SEISMIC ANALYSIS OF MULTISTOREY BUILDING WITH FLOATING COLUMN

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology
In
Structural Engineering

By

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Rourkela- 769008

MAY 2012

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DEPARTMENT OF CIVIL ENGINEERING
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CERTIFICATE

This is to certify that the thesis entitled, ***"SEISMIC ANALYSIS OF MULTISTOREY BUILDING WITH FLOATING COLUMN"*** *submitted by* ***SUKUMAR BEHERA*** *bearing roll no.* ***210CE2261*** *in partial fulfilment of the requirements for the award of* ***Master of Technology*** *degree in* ***Civil Engineering*** *with specialization in* ***"Structural Engineering"*** *during 2010-2012 session at the National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance.*

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

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ACKNOWLEDGEMENT

It is with a feeling of great pleasure that I would like to express my most sincere heartfelt gratitude to my guides, **Prof. A V Asha** and **Prof. K C Biswal**, professors, Dept. of Civil Engineering, NIT, Rourkela for their encouragement, advice, mentoring and research support throughout my studies. Their technical and editorial advice was essential for the completion of this dissertation. Their ability to teach, depth of knowledge and ability to achieve perfection will always be my inspiration.

I express my sincere thanks to **Prof. S. K. Sarangi**, Director of NIT, Rourkela & **Prof. N Roy**, Professor and HOD, Dept. of Civil Engineering NIT, Rourkela for providing me the necessary facilities in the department.

I would also take this opportunity to express my gratitude and sincere thanks to **Prof. P Sarakar,** my faculty and adviser and all faculty members of structural engineering, **Prof. M. R. Barik**, **Prof. S K Sahu** for their invaluable advice, encouragement, inspiration and blessings during the project.

I would like to express my eternal gratitude to **Er. S C Choudhury**, a M.Tech(Res) student, Dept. of Civil Engineering, NIT ,Rourkela for his enormous support, encouragement and advices. I would like to thank all my friends; they really were at the right place at the right time when I needed. I would also express my sincere thanks to laboratory Members of Department of Civil Engineering, NIT, Rourkela.

Last but not the least I would like to thank my parent, who taught me the value of hard work by their own example. I would like to share this bite of happiness with my father **Mr Gopinath Behera**, my mother **Mrs Sarada Behera** and my brother **Mr Sunil Behera(IITR)**. They rendered me enormous support during the whole tenure of my stay at NIT, Rourkela.

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ABSTRACT

In present scenario buildings with floating column is a typical feature in the modern multistory construction in urban India. Such features are highly undesirable in building built in seismically active areas. This study highlights the importance of explicitly recognizing the presence of the floating column in the analysis of building. Alternate measures, involving stiffness balance of the first storey and the storey above, are proposed to reduce the irregularity introduced by the floating columns.

FEM codes are developed for 2D multi storey frames with and without floating column to study the responses of the structure under different earthquake excitation having different frequency content keeping the PGA and time duration factor constant. The time history of floor displacement, inter storey drift, base shear, overturning moment are computed for both the frames with and without floating column.

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NOMENCLATURE

The principal symbols used in this thesis are presented for easy reference. A symbol is used for different meaning depending on the context and defined in the text as they occur.

English notation	Description
A	Area of the beam element
A_{max}	Maximum amplitude of acceleration of sinusoidal load
Ä	Sinusoidal acceleration loading
c	Damping of a single DOF system
[C]	Global damping matrix of the structure
$\underline{d}_0$, $\underline{\dot{d}}_0$, $\underline{\ddot{d}}_0$	Displacement, velocity, acceleration at time t=0 used in Newmark's Beta method
$\underline{d}_{i+1}$, $\underline{\dot{d}}_{i+1}$, $\underline{\ddot{d}}_{i+1}$	Displacement, velocity, acceleration at i^{th} time step used in Newmarks Beta method
E	Young's Modulus of the frame material
F0	Maximum displacement amplitude of sinusoidal load
F(t)	Force vector.
$F(t)_I$, $F(t)_D$, $F(t)_S$	Inertia, damping and stiffness component of reactive force.
K	Stiffness of a single DOF system
k^e	Stiffness matrix of a beam element
$[K^e]$	Transformed stiffness matrix of a beam element
[K]	Global stiffness matrix of the structure.
L	Length of the beam element
m	Mass of a single DOF system.
m_L^e	Lumped mass matrix
m^e	Consistent mass matrix of a beam element

$[M^e]$	Transformed consistent mass matrix of a beam element
$[M]$	Global mass matrix of structure
t	Time
$[T]$	Transformation matrix
$u(t)$	Displacement of a single DOF system
$\dot{u}(t)$	Velocity of a single DOF system
$\ddot{u}(t)$	Acceleration of a single DOF system
$U(t)$	Absolute nodal displacement.
$\dot{U}(t)$	Absolute nodal velocity.
$\ddot{U}(t)$	Absolute nodal acceleration.
$\ddot{U}_g(t)$	Ground acceleration due to earthquake.
ρ	Density of the beam material
β, γ	Parameters used in Newmarks Beta method
Δt	Time step used in Newmarks Beta method
μ	Mass ratio of secondary to primary system in 2 DOF system
ω	Sinusoidal forcing frequency
ζ	Damping ratio

CHAPTER 1

INTRODUCTION

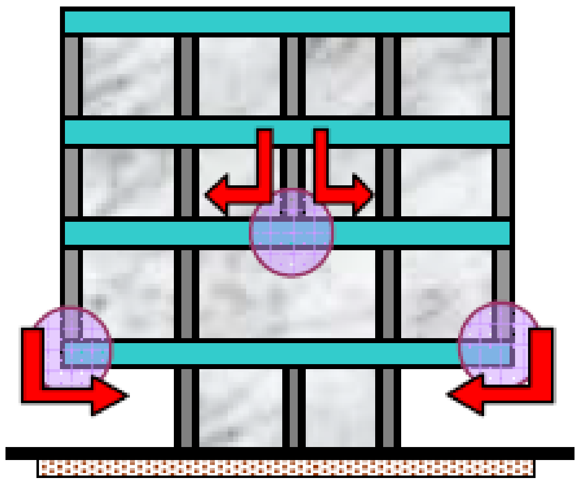
1.1 Introduction

Many urban multistorey buildings in India today have open first storey as an unavoidable feature. This is primarily being adopted to accommodate parking or reception lobbies in the first storey. Whereas the total seismic base shear as experienced by a building during an earthquake is dependent on its natural period, the seismic force distribution is dependent on the distribution of stiffness and mass along the height.

The behavior of a building during earthquakes depends critically on its overall shape, size and geometry, in addition to how the earthquake forces are carried to the ground. The earthquake forces developed at different floor levels in a building need to be brought down along the height to the ground by the shortest path; any deviation or discontinuity in this load transfer path results in poor performance of the building. Buildings with vertical setbacks (like the hotel buildings with a few storey wider than the rest) cause a sudden jump in earthquake forces at the level of discontinuity. Buildings that have fewer columns or walls in a particular storey or with unusually tall storey tend to damage or collapse which is initiated in that storey. Many buildings with an open ground storey intended for parking collapsed or were severely damaged in Gujarat during the 2001 Bhuj earthquake. Buildings with columns that hang or float on beams at an intermediate storey and do not go all the way to the foundation, have discontinuities in the load transfer path.

1.2 What is floating column

A column is supposed to be a vertical member starting from foundation level and transferring the load to the ground. The term floating column is also a vertical element which (due to architectural design/ site situation) at its lower level (termination Level) rests on a beam which is a horizontal member. The beams in turn transfer the load to other columns below it.



Hanging or Floating Columns

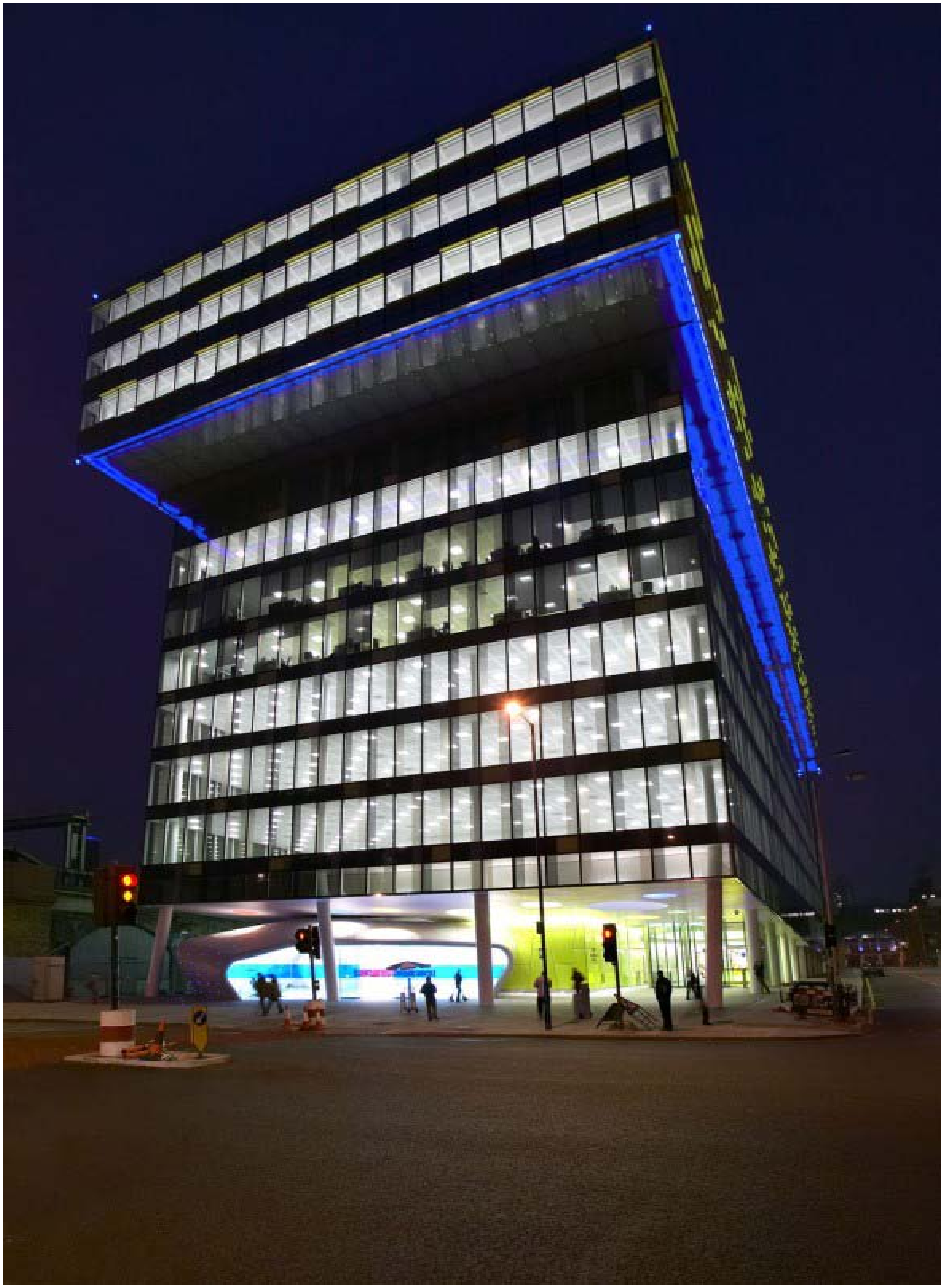
There are many projects in which floating columns are adopted, especially above the ground floor, where transfer girders are employed, so that more open space is available in the ground floor. These open spaces may be required for assembly hall or parking purpose. The transfer girders have to be designed and detailed properly, especially in earth quake zones. The column is a concentrated load on the beam which supports it. As far as analysis is concerned, the column is often assumed pinned at the base and is therefore taken as a point load on the transfer beam. STAAD Pro, ETABS and SAP2000 can be used to do the analysis of this type of structure. Floating columns are competent enough to carry gravity loading but transfer girder must be of adequate dimensions (Stiffness) with very minimal deflection.

Looking ahead, of course, one will continue to make buildings interesting rather than monotonous. However, this need not be done at the cost of poor behavior and earthquake safety of buildings. Architectural features that are detrimental to earthquake response of buildings should be avoided. If not, they must be minimized. When irregular features are included in buildings, a considerably higher level of engineering effort is required in the structural design and yet the building may not be as good as one with simple architectural features.

Hence, the structures already made with these kinds of discontinuous members are endangered in seismic regions. But those structures cannot be demolished, rather study can be done to strengthen the structure or some remedial features can be suggested. The columns of the first storey can be made stronger, the stiffness of these columns can be increased by retrofitting or these may be provided with bracing to decrease the lateral deformation.

Some pictures showing the buildings built with floating columns:

240 Park Avenue South in New York, United States



Palestra in London, United Kingdom

Chongqing Library in Chongqing, China

One-Housing-Group-by-Stock-Woolstencroft-in-London-United-Kingdom

1.3 Objective and scope of present work

The objective of the present work is to study the behavior of multistory buildings with floating columns under earthquake excitations.

Finite element method is used to solve the dynamic governing equation. Linear time history analysis is carried out for the multistory buildings under different earthquake loading of varying frequency content. The base of the building frame is assumed to be fixed. Newmark's direct integration scheme is used to advance the solution in time.

1.4 Organization

Presentation of the research effort is organized as follows:

- Chapter 2 presents the literature survey on seismic analysis of multi storey frame structures.
- Chapter 3 presents some theory and formulations used for developing the FEM program.
- Chapter 4 presents the validation of the FEM program developed and prediction of response of structure under different earthquake response.
- Chapter 5 concludes the present work. An account of possible scope of extension to the present study has been appended to the concluding remarks.
- Some important publication and books referred during the present investigation have been listed in the references.

CHAPTER 2

REVIEW OF LITERATURES

Current literature survey includes earthquake response of multi storey building frames with usual columns. Some of the literatures emphasized on strengthening of the existing buildings in seismic prone regions.

Maison and Neuss [15], (1984), Members of ASCE have preformed the computer analysis of an existing forty four story steel frame high-rise Building to study the influence of various modeling aspects on the predicted dynamic properties and computed seismic response behaviours. The predicted dynamic properties are compared to the building's true properties as previously determined from experimental testing. The seismic response behaviours are computed using the response spectrum (Newmark and ATC spectra) and equivalent static load methods.

Also**, Maison and Ventura** [16], (1991), Members of ASCE computed dynamic properties and response behaviours OF THIRTEEN-STORY BUILDING and this result are compared to the true values as determined from the recorded motions in the building during two actual earthquakes and shown that state-of-practice design type analytical models can predict the actual dynamic properties.

Arlekar, Jain & **Murty** [2], (1997) said that such features were highly undesirable in buildings built in seismically active areas; this has been verified in numerous experiences of strong shaking during the past earthquakes. They highlighted the importance of explicitly recognizing the presence of the open first storey in the analysis of the building, involving stiffness balance of the open first storey and the storey above, were proposed to reduce the irregularity introduced by the open first storey.

Awkar and Lui [3], (1997) studied responses of multi-story flexibly connected frames subjected to earthquake excitations using a computer model. The model incorporates connection flexibility as well as geometrical and material nonlinearities in the analyses and concluded that the study indicates that connection flexibility tends to increase upper stories' inter-storey drifts but reduce base shears and base overturning moments for multi-story frames.

Balsamoa, Colombo, Manfredi, Negro & Prota [4] (2005) performed pseudodynamic tests on an RC structure repaired with CFRP laminates. The opportunities provided by the use of Carbon Fiber Reinforced Polymer (CFRP) composites for the seismic repair of reinforced concrete (RC) structures were assessed on a full-scale dual system subjected to pseudodynamic tests in the ELSA laboratory. The aim of the CFRP repair was to recover the structural properties that the frame had before the seismic actions by providing both columns and joints with more deformation capacity. The repair was characterized by a selection of different fiber textures depending on the main mechanism controlling each component. The driving principles in the design of the CFRP repair and the outcomes of the experimental tests are presented in the paper. Comparisons between original and repaired structures are discussed in terms of global and local performance. In addition to the validation of the proposed technique, the experimental results will represent a reference database for the development of design criteria for the seismic repair of RC frames using composite materials.

Vasilopoulos and Beskos [23], (2006) performed rational and efficient seismic design methodology for plane steel frames using advanced methods of analysis in the framework of Eurocodes 8 and 3 . This design methodology employs an advanced finite element method of analysis that takes into account geometrical and material nonlinearities and member and frame imperfections. It can sufficiently capture the limit states of displacements, strength, stability and damage of the structure.

Bardakis & Dritsos [5] (2007) evaluated the American and European procedural assumptions for the assessment of the seismic capacity of existing buildings via pushover analyses. The FEMA and the Euro code-based GRECO procedures have been followed in order to assess a four-storeyed bare framed building and a comparison has been made with available experimental results.

Mortezaei *et al* [17] (2009) recorded data from recent earthquakes which provided evidence that ground motions in the near field of a rupturing fault differ from ordinary ground motions, as they can contain a large energy, or ‘‘directivity” pulse. This pulse can cause considerable damage during an earthquake, especially to structures with natural periods close to those of the pulse. Failures of modern engineered structures observed within the near-fault region in recent earthquakes have revealed the vulnerability of existing RC buildings against pulse-type ground motions. This may be due to the fact that these modern structures had been designed primarily using the design spectra of available standards, which have been developed using stochastic processes with relatively long duration that characterizes more distant ground motions. Many recently designed and constructed buildings may therefore require strengthening in order to perform well when subjected to near-fault ground motions. Fiber Reinforced Polymers are considered to be a viable alternative, due to their relatively easy and quick installation, low life cycle costs and zero maintenance requirements.

Ozyigit [19], (2009) performed free and forced in-plane and out-of-plane vibrations of frames are investigated. The beam has a straight and a curved part and is of circular cross section. A concentrated mass is also located at different points of the frame with different mass ratios. FEM is used to analyze the problem.

Williams, Gardoni & Bracci [24] (2009) studied the economic benefit of a given retrofit procedure using the framework details. A parametric analysis was conducted to determine how certain parameters affect the feasibility of a seismic retrofit. A case study was performed for the example buildings in Memphis and San Francisco using a modest retrofit procedure. The results of the parametric analysis and case study advocate that, for most situations, a seismic retrofit of an existing building is more financially viable in San Francisco than in Memphis.

Garcia *et al* [10] (2010) tested a full-scale two-storey RC building with poor detailing in the beam column joints on a shake table as part of the European research project ECOLEADER. After the initial tests which damaged the structure, the frame was strengthened using carbon fibre reinforced materials (CFRPs) and re-tested. This paper investigates analytically the efficiency of the strengthening technique at improving the seismic behaviour of this frame structure. The experimental data from the initial shake table tests are used to calibrate analytical models. To simulate deficient beam_column joints, models of steel_concrete bond slip and bond-strength degradation under cyclic loading were considered. The analytical models were used to assess the efficiency of the CFRP rehabilitation using a set of medium to strong seismic records. The CFRP strengthening intervention enhanced the behaviour of the substandard beam_column joints, and resulted in substantial improvement of the seismic performance of the damaged RC frame. It was shown that, after the CFRP intervention, the damaged building would experience on average 65% less global damage compared to the original structure if it was subjected to real earthquake excitations.

Niroomandi, Maheri, Maheri & Mahini [18] (2010) retrofitted an eight-storey frame strengthened previously with a steel bracing system with web- bonded CFRP. Comparing the seismic performance of the FRP retrofitted frame at joints with that of the steel X-braced retrofitting method, it was concluded that both retrofitting schemes have comparable abilities to increase the ductility reduction factor and the over-strength factor; the former comparing better on ductility and the latter on over-strength. The steel bracing of the RC frame can be beneficial if a substantial increase in the stiffness and the lateral load resisting capacity is required. Similarly, FRP retrofitting at joints can be used in conjunction with FRP retrofitting of beams and columns to attain the desired increases.

CHAPTER 3

FINITE ELEMENT FORMULATION

The finite element method (FEM), which is sometimes also referred as finite element analysis (FEA), is a computational technique which is used to obtain the solutions of various boundary value problems in engineering, approximately. Boundary value problems are sometimes also referred to as field value problems. It can be said to be a mathematical problem wherein one or more dependent variables must satisfy a differential equation everywhere within the domain of independent variables and also satisfy certain specific conditions at the boundary of those domains. The field value problems in FEM generally has field as a domain of interest which often represent a physical structure. The field variables are thus governed by differential equations and the boundary values refer to the specified value of the field variables on the boundaries of the field. The field variables might include heat flux, temperature, physical displacement, and fluid velocity depending upon the type of physical problem which is being analyzed.

3.1 Static analysis

3.1.1 Plane frame element

The plane frame element is a two-dimensional finite element with both local and global coordinates. The plane frame element has modulus of elasticity E, moment of inertia I, cross-sectional area A, and length L. Each plane frame element has two nodes and is inclined with an angle of θ measured counterclockwise from the positive global X axis as shown in figure. Let C= cosθ and S= sinθ.

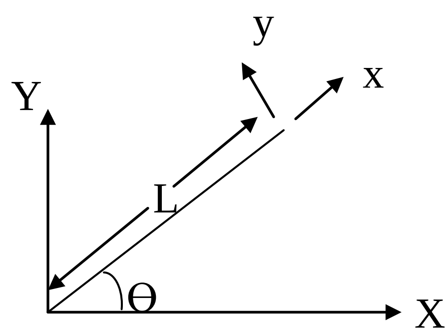


Fig. 3.1 The plane frame element

It is clear that the plane frame element has six degree of freedom – three at each node (two displacements and a rotation). The sign convention used is that displacements are positive if they point upwards and rotations are positive if they are counterclockwise. Consequently for a structure with n nodes, the global stiffness matrix K will be 3n X 3n (since we have three degrees of freedom at each node). The global stiffness matrix K is assembled by making calls to the MATLAB function PlaneFrameAssemble which is written specially for this purpose.

Once the global stiffness matrix K is obtained we have the following structure equation:

$$[K]\{U\} = \{F\} \tag{3.1}$$

Where [K] is stiffness matrix, {U} is the global nodal displacement vector and {F} is the global nodal force vector. At this step boundary conditions are applied manually to the vectors U and F. Then the matrix equation (3.1) is solved by partitioning and Gaussian elimination. Finally once the unknown displacements and reactions are found, the nodal force vector is obtained for each element as follows:

$$\{f\} = [k]\,[R]\,\{u\} \tag{3.2}$$

Where {f} is the 6 X 1 nodal force vector in the element and {u} is the 6 X 1 element displacement vector. The matrices [k] and [R] are given by the following:

$$[k] = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & \frac{-EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{6EI}{L^2} & \frac{2EI}{L} \\ \frac{-EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \tag{3.3}$$

$$[R] = \begin{bmatrix} C & S & 0 & 0 & 0 & 0 \\ -S & C & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & C & S & 0 \\ 0 & 0 & 0 & -S & C & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{3.4}$$

The first and second element in the vector {u} are the two displacements while the third element is the rotation, respectively, at the first node, while the fourth and fifth element are the two displacements while the sixth element is the rotation, respectively, at the second node.

3.1.2 Steps followed for the analysis of frame

1. **Discretising the domain:** Dividing the element into number of nodes and numbering them globally i;e breaking down the domain into smaller parts.

2. **Writing of the Element stiffness matrices:** The element stiffness matrix or the local stiffness matrix is found for all elements and the global stiffness matrix of size 3n x 3n is assembled using these local stiffness matrices.
3. **Assembling the global stiffness matrices:** The element stiffness matrices are combined globally based on their degrees of freedom values.
4. **Applying the boundary condition:** The boundary element condition is applied by suitably deleting the rows and columns which are not of our interest.
5. **Solving the equation:** The equation is solved in MATLAB to give the value of U.
6. **Post- processing**: The reaction at the support and internal forces are calculated.

3.2 Dynamic analysis

Dynamic analysis of structure is a part of structural analysis in which behavior of flexible structure subjected to dynamic loading is studied. Dynamic load always changes with time. Dynamic load comprises of wind, live load, earthquake load etc. Thus in general we can say almost all the real life problems can be studied dynamically.

If dynamic loads changes gradually the structure’s response may be approximately by a static analysis in which inertia forces can be neglected. But if the dynamic load changes quickly, the response must be determined with the help of dynamic analysis in which we cannot neglect inertial force which is equal to mass time of acceleration (Newton’s 2nd law).

Mathematically F = M x a

Where F is inertial force, M is inertial mass and ‘a’ is acceleration.

Furthermore, dynamic response (displacement and stresses) are generally much higher than the corresponding static displacements for same loading amplitudes, especially at resonant conditions.

The real physical structures have many numbers of displacement. Therefore the most critical part of structural analysis is to create a computer model, with the finite number of mass less member and finite number of displacement of nodes which simulates the real behavior of structures. Another difficult part of dynamic analysis is to calculate energy dissipation and to boundary condition. So it is very difficult to analyze structure for wind and seismic load. This difficulty can be reduced using various programming techniques. In our project we have used finite element analysis and programmed in MATLAB.

3.2.1 Time history analysis

A linear time history analysis overcomes all the disadvantages of modal response spectrum analysis, provided non-linear behavior is not involved. This method requires greater computational efforts for calculating the response at discrete time. One interesting advantage of such procedure is that the relative signs of response qualities are preserved in the response histories. This is important when interaction effects are considered in design among stress resultants.

Here dynamic response of the plane frame model to specified time history compatible to IS code spectrum and Elcentro (EW) has been evaluated.

The equation of motion for a multi degree of freedom system in matrix form can be expressed as

$$[m]\{\ddot{x}\} + [c]\{\dot{x}\} + [k]\{x\} = -\ddot{x}_g(t)[m]\{I\} \qquad (3.5)$$

Where,

$[m]$= mass matrix

$[k]$= stiffness matrix

$[c]$= damping matrix

$\{I\}$= unit vector

$\ddot{x_g}(t)$= ground acceleration

The mass matrix of each element in global direction can be found out using following expression:

$$m = [T^T]\,[m_e]\,[T] \tag{3.6}$$

$$[m_e] = \frac{\rho\, A\, L}{420} \begin{bmatrix} 140 & 0 & 0 & 70 & 0 & 0 \\ 0 & 156 & 22L & 0 & 54 & -13L \\ 0 & 22L & 4L^2 & 0 & 13L & -3L^2 \\ 70 & 0 & 0 & 140 & 0 & 0 \\ 0 & 54 & 13L & 0 & 156 & -22L \\ 0 & -13L & -3L^2 & 0 & -22L & 4L^2 \end{bmatrix} \tag{3.7}$$

$$[T] = \begin{bmatrix} C & S & 0 & 0 & 0 & 0 \\ -S & C & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & C & S & 0 \\ 0 & 0 & 0 & -S & C & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

The solution of equation of motion for any specified forces is difficult to obtain, mainly due to due to coupling variables {x} in the physical coordinate. In mode superposition analysis or a modal analysis a set of normal coordinates i.e principal coordinate is defined, such that, when expressed in those coordinates, the equations of motion becomes uncoupled. The physical coordinate {x} may be related with normal or principal coordinates {q} from the transformation expression as,

$$\{ x \} = [\Phi] \{ q \}$$

[Φ] is the modal matrix

Time derivative of $\{ x \}$ are,

$$\{\dot{x}\} = [\Phi] \{\dot{q}\}$$

$$\{\ddot{x}\} = [\Phi] \{\ddot{q}\}$$

Substituting the time derivatives in the equation of motion, and pre-multiplying by $[\Phi]^T$ results in,

$$[\Phi]^T[m][\Phi]\{\ddot{q}\} + [\Phi]^T[c][\Phi]\{\dot{q}\} + [\Phi]^T[k][\Phi]\{q\} = (-\ddot{x}_g(t)[\Phi]^T[m]\{I\}) \qquad (3.8)$$

More clearly it can be represented as follows:

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{P_{eff}(t)\} \qquad (3.9)$$

Where,

$[M]= [\Phi]^T[m][\Phi]$

$[C]= [\Phi]^T[c][\Phi] = 2\,\zeta\,[M]\,[\omega]$

$[K]= [\Phi]^T[k][\Phi]$

$\{P_{eff}(t)\}= (-\ddot{x}_g(t)[\Phi]^T[m]\{I\})$

$[M]$, $[C]$ and $[K]$ are the diagonalised modal mass matrix, modal damping matrix and modal stiffness matrix, respectively, and {P_eff(t)} is the effective modal force vector.

3.2.2 Newmark's method

Newmark's numerical method has been adopted to solve the equation 3.9. Newmark's equations are given by

$$\underline{\dot{d}}_{i+1} = \underline{\dot{d}}_i + (\Delta t)\left[(1-\gamma)\underline{\ddot{d}}_i + \gamma\underline{\ddot{d}}_{i+1}\right] \qquad (3.10)$$

$$\underline{\dot{d}}_{i+1} = \underline{\dot{d}}_i + (\Delta t)\underline{\dot{d}}_i + (\Delta t)^2\left[\left(\frac{1}{2}-\beta\right)\underline{\ddot{d}}_i + \beta\underline{\ddot{d}}_{i+1}\right] \qquad (3.11)$$

Where β and γ are parameters chosen by the user. The parameter β is generally chosen between 0 and ¼, and γ is often taken to be ½. For instance, choosing γ = ½ and β = 1/6, are chosen, eq. 4.12 and eq. 4.13 correspond to those foe which a linear acceleration assumption is valid within each time interval. For γ = ½ and β =¼, it has been shown that the nu merical analysis is stable; that is, computed quantities such as displacement and velocities do not become unbounded regardless of the time step chosen.

To find $\underline{d}_{i+1}$, we first multiply eq. 4.13 by the mass matrix $\underline{M}$ and then substitute the value of $\underline{\ddot{d}}_{i+1}$ into this eq. to obtain

$$\underline{M}\,\underline{\ddot{d}}_{i+1} = \underline{M}\,\underline{d}_i + (\Delta t)\underline{M}\,\underline{\dot{d}}_i + (\Delta t)^2\underline{M}\left(\frac{1}{2}-\beta\right)\underline{\ddot{d}}_i + \beta(\Delta t)^2\left[\underline{F}_{i+1} - \underline{K}\,\underline{d}_{i+1}\right] \qquad (3.12)$$

Combining the like terms of eq. 4.14 we obtain

$$\left(\underline{M} + \beta(\Delta t)^2\underline{K}\right)\underline{d}_{i+1} = \beta(\Delta t)^2\underline{F}_{i+1} + \underline{M}\,\underline{d}_i + (\Delta t)\underline{M}\,\underline{\dot{d}}_i + (\Delta t)^2\underline{M}\left(\frac{1}{2}-\beta\right)\underline{\ddot{d}}_i \qquad (3.13)$$

Finally, dividing above eq. by $\beta(\Delta t)^2$, we obtain

$$\underline{K}'\underline{d}_{i+1} = \underline{F}'_{i+1} \tag{3.14}$$

$$\underline{K}' = \underline{K} + \frac{1}{\beta\,(\Delta t)^2}\underline{M} \tag{3.15}$$

$$\underline{F}'_{i+1} = \underline{F}_{i+1} + \frac{\underline{M}}{\beta(\Delta t)^2}\left[\underline{d}_i + (\Delta t)\underline{\dot{d}}_i + \left(\frac{1}{2} - \beta\right)(\Delta t)^2\underline{\ddot{d}}_i\right] \tag{3.16}$$

The solution procedure using Newmark's equations is as follows:

1. Starting at time t=0, $\underline{d}_0$ is known from the given boundary conditions on displacement, and $\underline{\dot{d}}_0$ is known from the initial velocity conditions.
2. **2.** Solve eq. 4.5 at t=0 for $\underline{\ddot{d}}_0$ (unless $\underline{\ddot{d}}_0$ is known from an initial acceleration condition); that is,

$$\underline{\ddot{d}}_0 = \underline{M}^{-1}\left(\underline{F}_0 - \underline{K}\underline{d}_0\right)$$

3. **3.** Solve eq. 4.16 for $\underline{d}_1$, because $\underline{F}'_{i+1}$ is known for all time steps and , $\underline{d}_0$, $\underline{\dot{d}}_0$, $\underline{\ddot{d}}_0$ are known from steps 1 and 2.
4. Use eq. 4.13 to solve for $\underline{\ddot{d}}_1$ as

$$\underline{\ddot{d}}_1 = \frac{1}{\beta(\Delta t)^2}\left[\underline{d}_1 - \underline{d}_0 - (\Delta t)\underline{\dot{d}}_0 - (\Delta t)^2\left(\frac{1}{2} - \beta\right)\underline{\ddot{d}}_0\right]$$

5. Solve eq. 4.12 directly for $\underline{\dot{d}}_1$
6. Using the results of steps 4 and 5, go back to step 3 to solve for $\underline{d}_2$ and then to steps 4 and 5 to solve for $\underline{\ddot{d}}_2$ and $\underline{\dot{d}}_2$. Use steps 3-5 repeatedly to solve for $\underline{d}_{i+1,}$ $\underline{\dot{d}}_{i+1}$ and $\underline{\ddot{d}}_{i+1}$.

CHAPTER 4

RESULT AND DISCUSSION

The behavior of building frame with and without floating column is studied under static load, free vibration and forced vibration condition. The finite element code has been developed in MATLAB platform.

4.1 Static analysis

A four storey two bay 2d frame with and without floating column are analyzed for static loading using the present FEM code and the commercial software *STAAD Pro*.

Example 4.1

The following are the input data of the test specimen:

Size of beam – 0.1 X 0.15 m

Size of column – 0.1 X 0.125 m

Span of each bay – 3.0 m

Storey height – 3.0 m

Modulus of Elasticity, E = 206.84 X 10^6 kN/m^2

Support condition – Fixed

Loading type – Live (3.0 kN at 3rd floor and 2 kN at 4th floor)

Fig. 4.1 and Fig.4.2 show the sketchmatic view of the two frame without and with floating column respectively. From Table 4.1 and 4.2, we can observe that the nodal displacement values obtained from present FEM in case of frame with floating column are more than the corresponding nodal displacement values of the frame without floating column. Table 4.3 and 4.4 show the nodal displacement value obtained from STAAD Pro of the frame without and with floating column respectively and the result are very comparable with the result obtained in present FEM.

. **Fig. 4.1** 2D Frame with usual columns

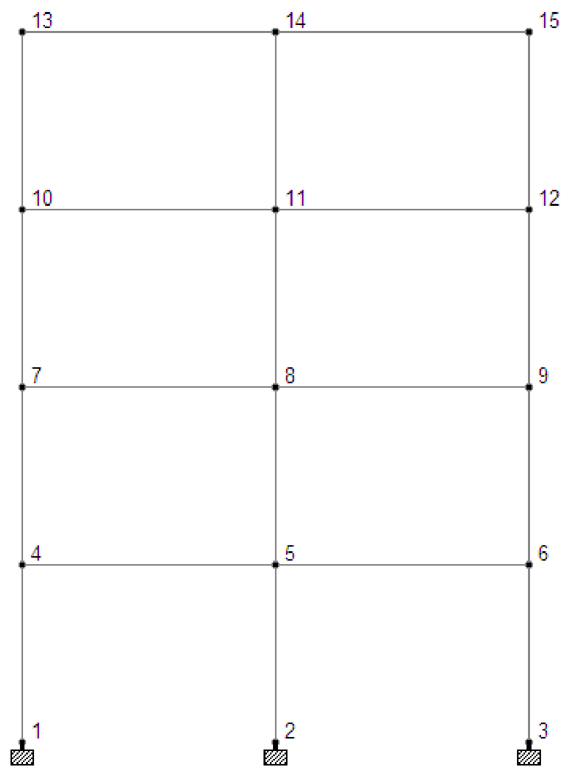


Fig.4.2 2D Frame with Floating column

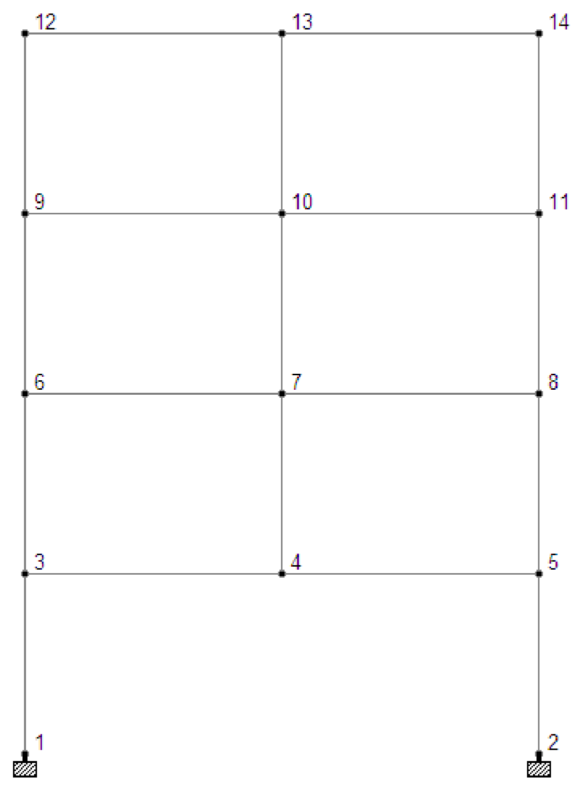


Table 4.1 Global deflection at each node for general frame obtained in present FEM

Node	Horizontal X mm	Vertical Y mm	Rotational rZ rad
1	0	0	0
2	0	0	0
3	0	0	0
4	1.6	0	0
5	1.6	0	0
6	1.6	0	0
7	3.8	0	0
8	3.8	0	0
9	3.8	0	0
10	5.8	0	0
11	5.8	0	0
12	5.8	0	0
13	6.7	0	0
14	6.7	0	0
15	6.7	0	0

Table 4.2 Global deflection at each node for general frame obtained in STAAD Pro.

Node	Horizontal X mm	Vertical Y mm	Rotational rZ rad
1	0	0	0
2	0	0	0
3	0	0	0
4	1.4	0	0
5	1.4	0	0
6	1.4	0	0
7	3.6	0	0
8	3.6	0	0
9	3.6	0	0
10	5.6	0	0
11	5.6	0	0
12	5.6	0	0
13	6.8	0	0
14	6.8	0	0
15	6.8	0	0

Table 4.3 Global deflection at each node for frame with floating column obtained in present FEM

Node	Horizontal	Vertical	Rotational
	X mm	Y mm	rZ rad
1	0	0	0
2	0	0	0
3	2.6	0	0
4	2.6	0	0
5	2.6	0	0
6	4.8	0	0
7	4.8	0	0
8	4.8	0	0
9	6.8	0	0
10	6.8	0	0
11	6.8	0	0
12	7.8	0	0
13	7.8	0	0
14	7.8	0	0

Table 4.4 Global deflection at each node for frame with floating column obtained in STAAD Pro

Node	Horizontal	Vertical	Rotational
	X mm	Y mm	rZ rad
1	0	0	0
2	0	0	0
3	2.6	0	0
4	2.6	0	0
5	2.6	0	0
6	4.8	0	0
7	4.8	0	0
8	4.8	0	0
9	6.8	0	0
10	6.8	0	0
11	6.8	0	0
12	7.7	0	0
13	7.7	0	0
14	7.7	0	0

4. 2 Free vibration analysis

Example 4.2

In this example a two storey one bay 2D frame is taken. Fig.4.3 shows the sketchmatic view of the 2D frame. The results obtained are compared with Maurice Petyt[21]. The input data are as follows:

Span of bay = 0.4572 m

Storey height = 0.2286 m

Size of beam = (0.0127 x 0.003175) m

Size of column = (0.0127 x 0.003175) m

Modulus of elasticity, E = 206.84 x10^6 kN/m^2

Density, ρ = 7.83 x 10^3 Kg/m^3

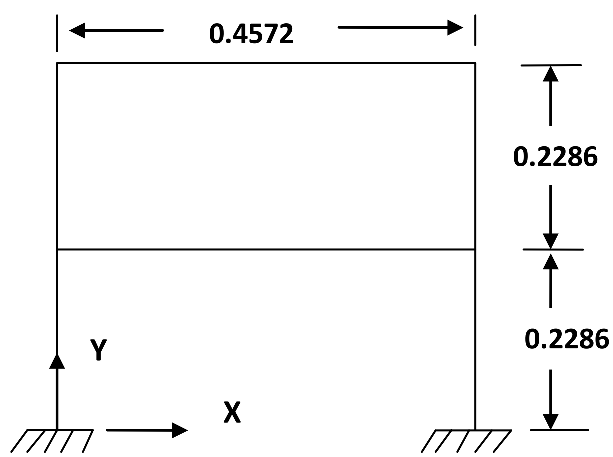


Fig. 4.3 Geometry of the 2 dimensional framework. Dimensions are in meter

Table 4.5 shows the value of free vibration frequency of the 2D frame calculated in present FEM. It is observed from Table 4.5 that the present results are in good agreement with the result given by Maurice Petyt [21].

Table 4.5 Free vibration frequency(Hz) of the 2D frame without floating column

Mode	Maurice Petyt [21]	Present FEM	% Variation
1	15.14	15.14	0.00
2	53.32	53.31	0.02
3	155.48	155.52	0.03
4	186.51	186.59	0.04
5	270.85	270.64	0.08

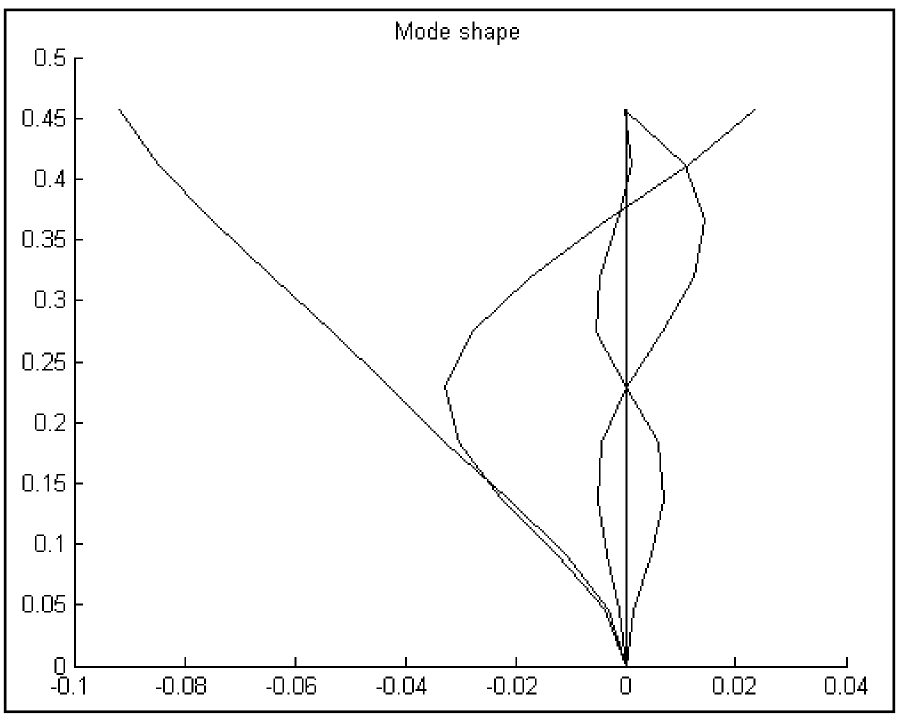


Fig. 4.4 Mode shape of the 2D framework

4. 3 Forced vibration analysis

Example 4.3

For the forced vibration analysis, a two bay four storey 2D steel frame is considered. The frame is subjected to ground motion, the compatible time history of acceleration as per spectra of IS 1893 (part 1): 2002.

The dimension and material properties of the frame is as follows:

Young's modulus. E= 206.84 x 10^6 kN/m^2

Density, ρ = 7.83 x10^3 Kg/m^3

Size of beam = (0.1 x 0.15) m

Size of column = (0.1 x 0.125) m

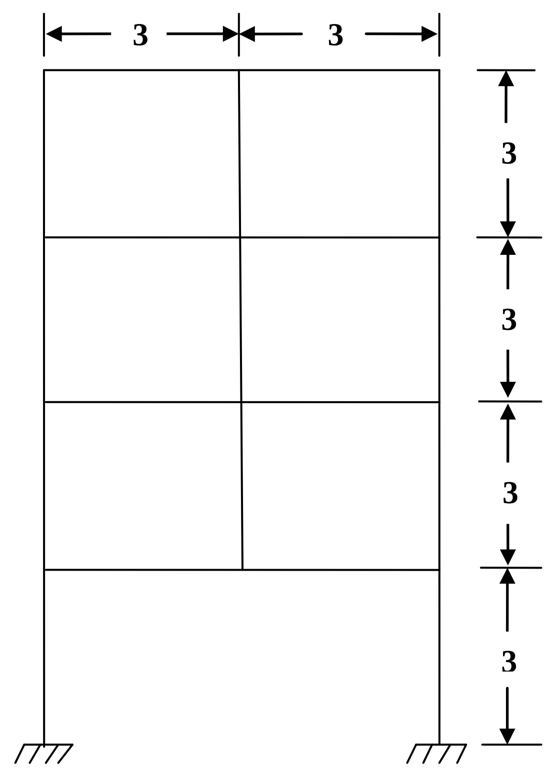


Fig. 4.5 Geometry of the 2 dimensional frame with floating column. Dimensions are in meter

Fig.4.6 shows the compatible time history as per spectra of IS 1893 (part 1): 2002. Fig.4.7 and 4.8 show the maximum top floor displacement of the 2D frame obtained in present FEM and STAAD Pro respectively.

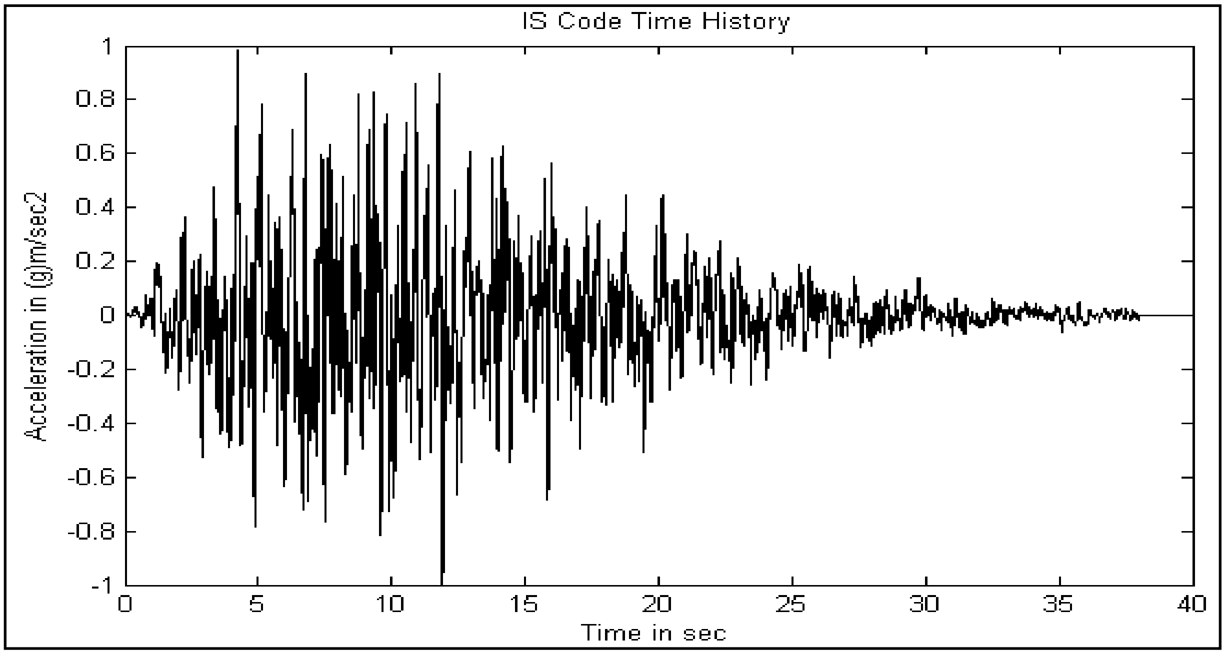


Fig. 4.6 Compatible time history as per spectra of IS 1893 (part 1): 2002

Free vibration frequencies of the 2D steel frame with floating column are presented in Table 4.6. In this table the values obtained in present FEM and STAAD Pro are compared. Table 4.7 shows the comparison of maximum top floor displacement of the frame obtained in present FEM and STAAD Pro which are in very close agreement.

Table 4.6 Comparison of predicted frequency (Hz) of the 2D steel frame with floating column obtained in present FEM and STAAD Pro.

Mode	STAAD Pro	Present FEM	% Variation
1	2.16	2.17	0.28
2	6.78	7.00	3.13
3	11.57	12.62	8.32
4	12.37	13.04	5.14

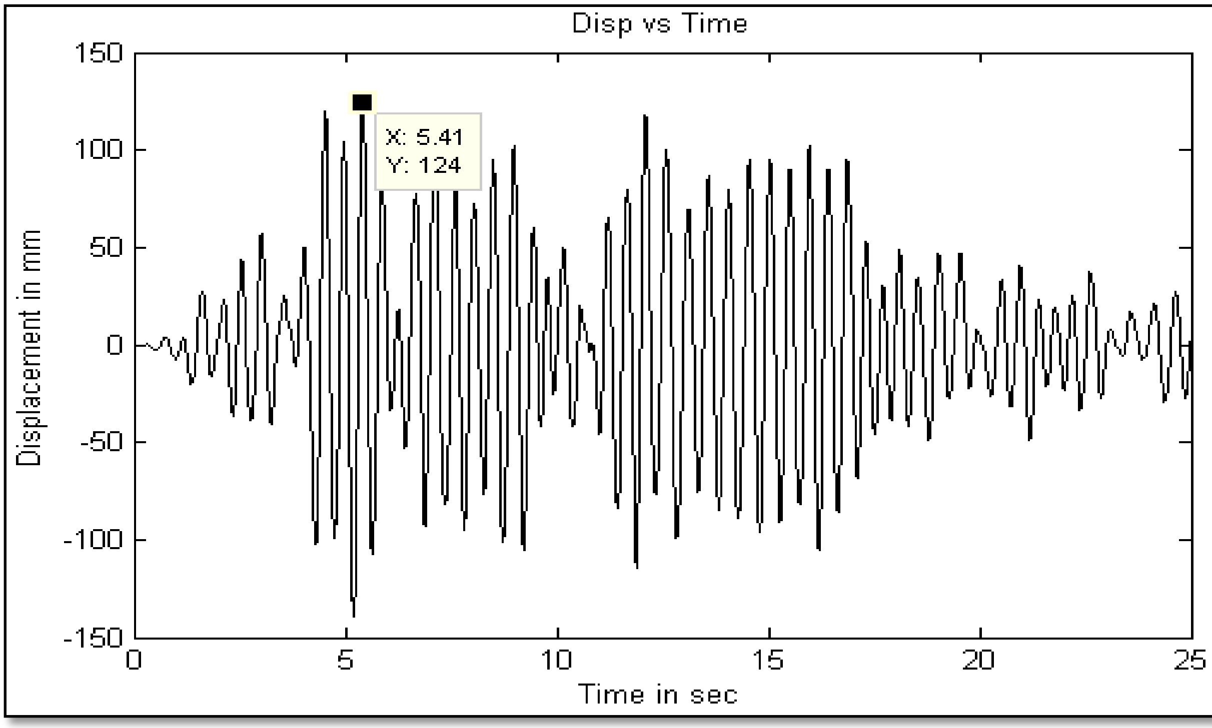


Fig. 4.7 Displacement vs time response of the 2D steel frame with floating column obtained in present FEM

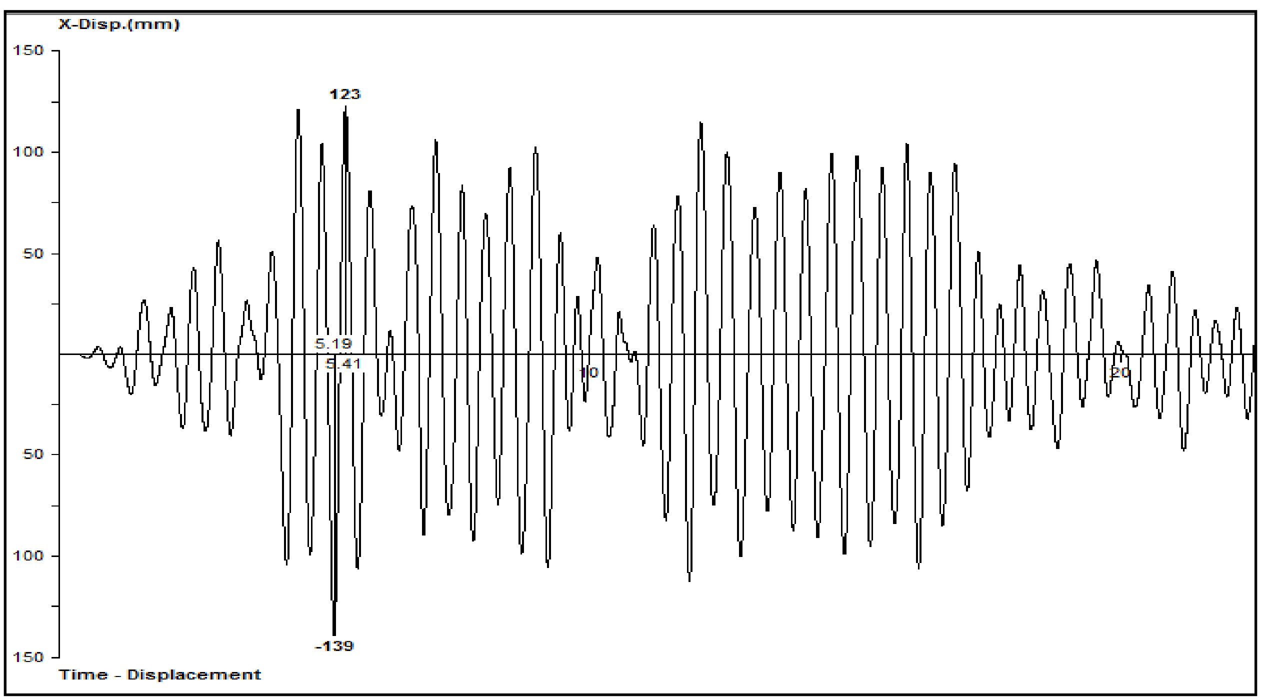


Fig. 4.8 Displacement vs time response of the 2D steel frame with floating column obtained in STAAD Pro

Table 4.7 Comparison of predicted maximum top floor displacement (mm) of the 2D steel frame with floating column in present FEM and STAAD Pro.

Maximum top floor displacement (mm)		% Variation
STAAD Pro.	**Present FEM**	
123	124	0.81

Example 4.4

The frame used in Example 4.3 is taken only by changing the material property and size of structural members. Size and material property of the structural members are as follows:

Size of beam = (0.25 x 0.3) m

Size of column = (0.25 x 0.25) m

Young's modulus, E= 22.36 x 10^9 N/m^2

Density, ρ = 2500 Kg/m^3

Fig.4.9 and 4.10 show the maximum top floor displacement of the 2D frame obtained in STAAD Pro and present FEM and respectively. Free vibration frequencies of the 2D concrete frame with floating column are presented in Table 4.8. In this table the values obtained in present FEM and STAAD Pro are compared. Table 4.9 shows the comparison of maximum top floor displacement of the frame obtained in present FEM and STAAD Pro which are in very close agreement.

Table 4.8 Comparison of predicted frequency(Hz) of the 2D concrete frame with floating column obtained in present FEM and STAAD Pro.

Mode	STAAD Pro	Present FEM	% Variation
1	2.486	2.52	1.37
2	7.78	8.09	3.98
3	13.349	14.67	9.89
4	13.938	14.67	5.25

Table 4.9 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with floating column obtained in present FEM and STAAD Pro.

Maximum top floor displacement		% Variation
STAAD Pro.	Present FEM	
118	121.2	2.71

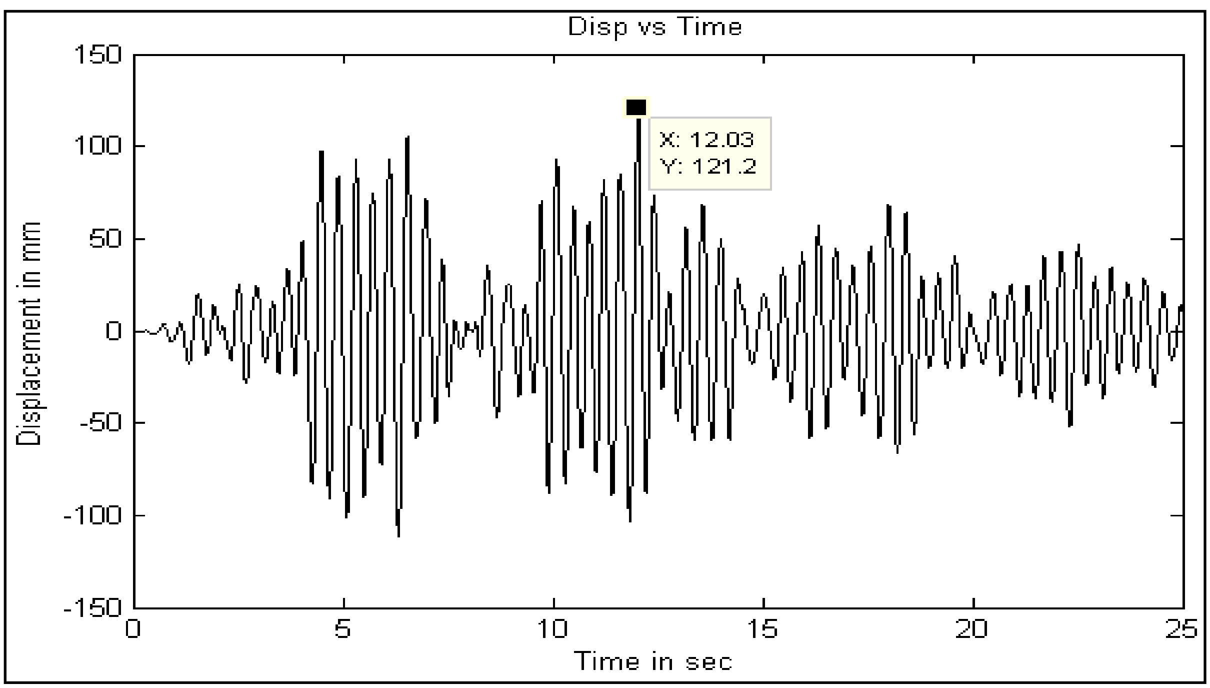


Fig. 4.10 Displacement vs time response of the 2D concrete frame with floating column plotted in present FEM

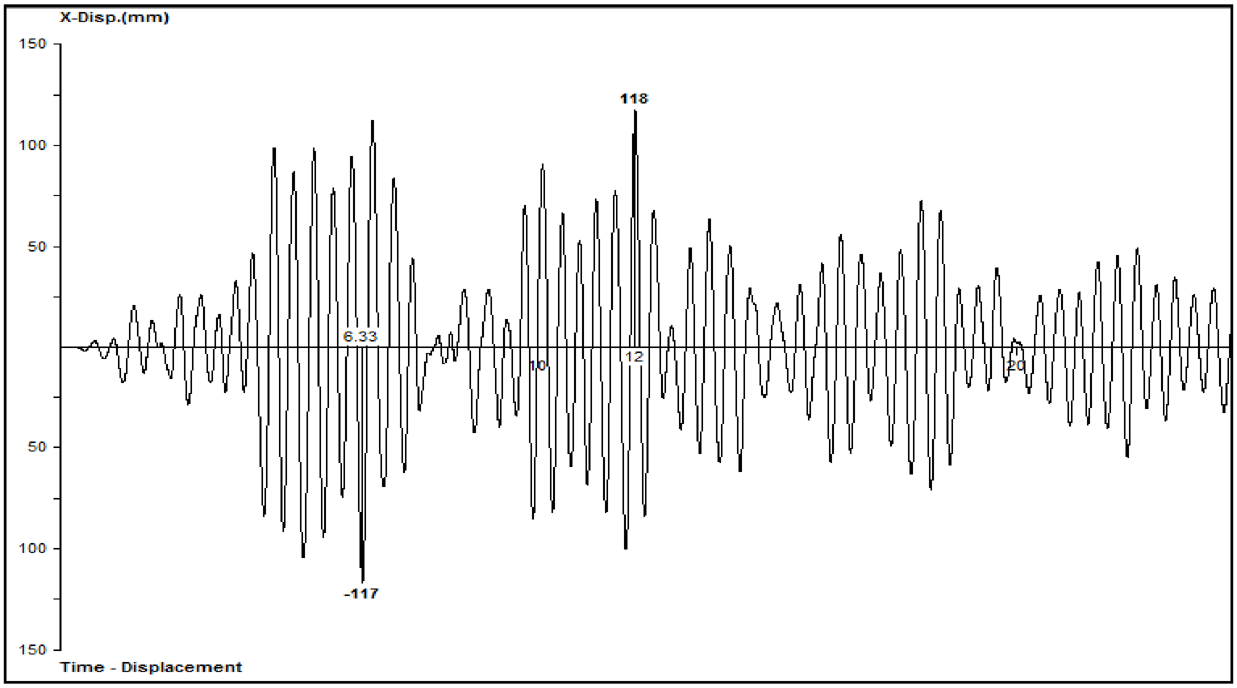


Fig. 4.9 Displacement vs time response of the 2D concrete frame with floating column given by STAAD Pro

Example 4.5

In this example two concrete frames with and without floating column having same material property and dimension are analyzed under same loading condition. Here "Compatible time history as per spectra of IS 1893 (part 1): 2002" is applied on the structures. IS code data is an intermediate frequency content data. IS code data has PGA value as 1.0g This frame is also analyzed under other earthquake data having different PGA value in further examples, hence it has scaled down to 0.2g. The section and material property for present study are as follows:

Young modulus, E= 22.36 x 10^6 kN/m^2, Density, ρ = 2500 Kg/m^3

Size of beam = (0.25 x 0.4) m, Size of column = (0.25 x 0.3) m

Storey height, h = 3.0m, Span = 3.0m

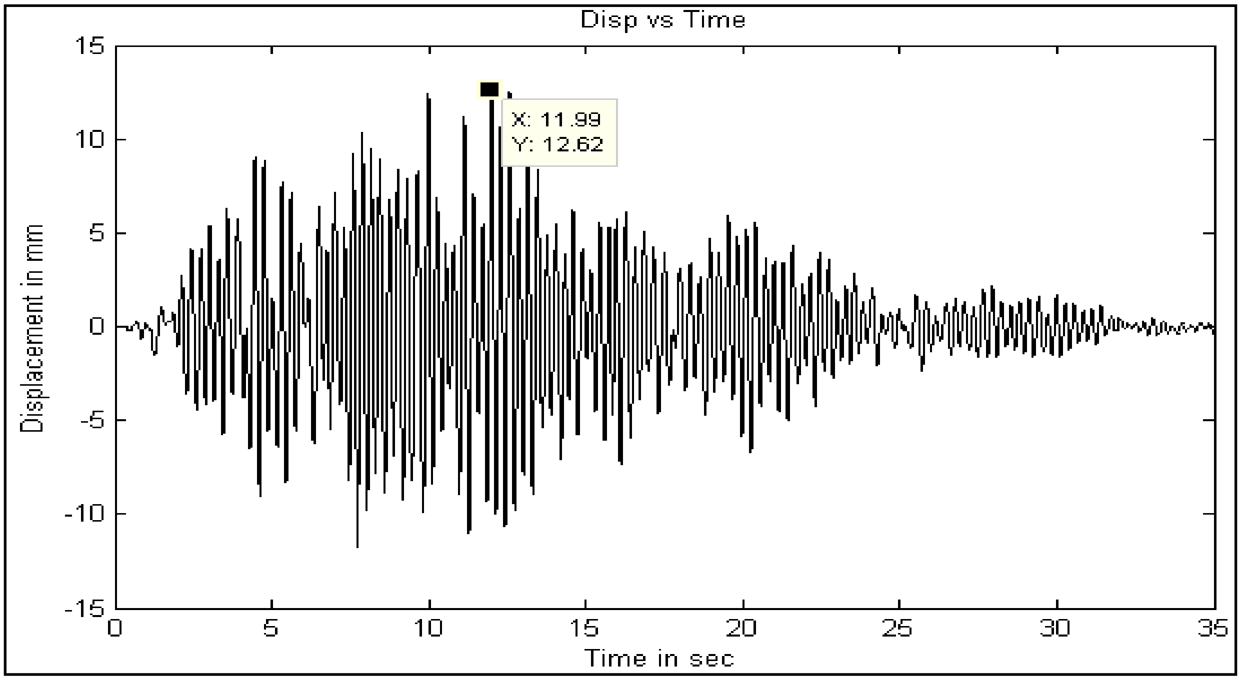


Fig. 4.11 Displacement vs time response of the 2D concrete frame without floating column under IS code time history excitation

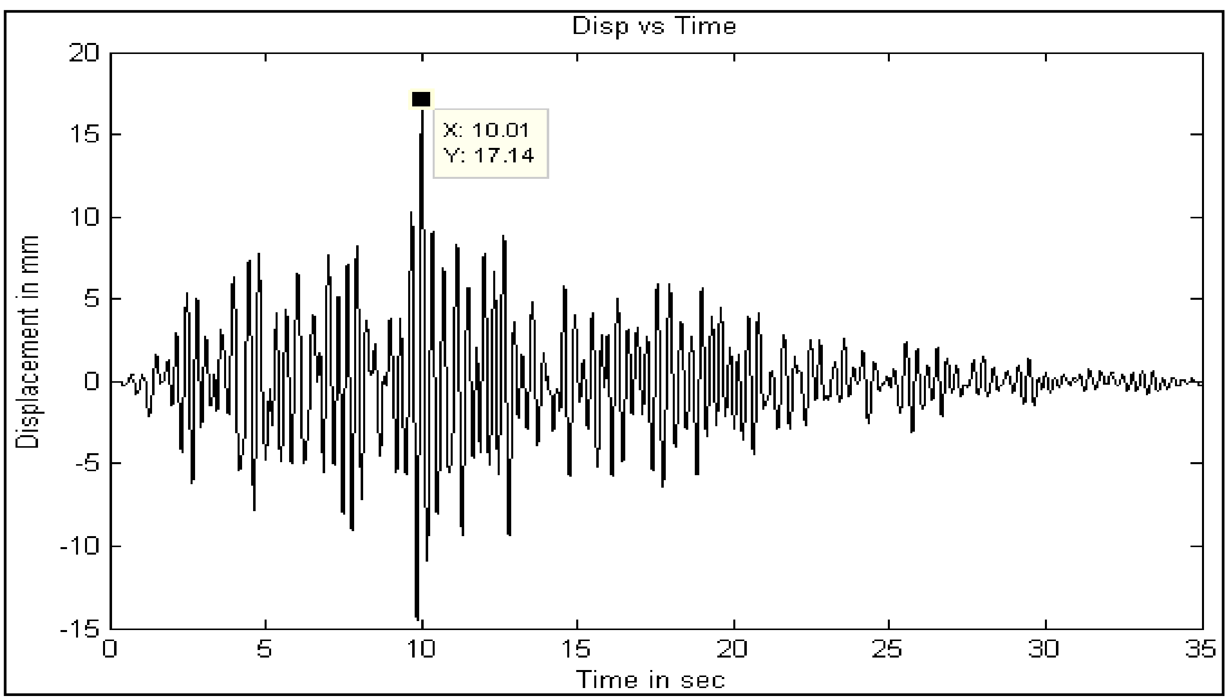


Fig. 4.12 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation

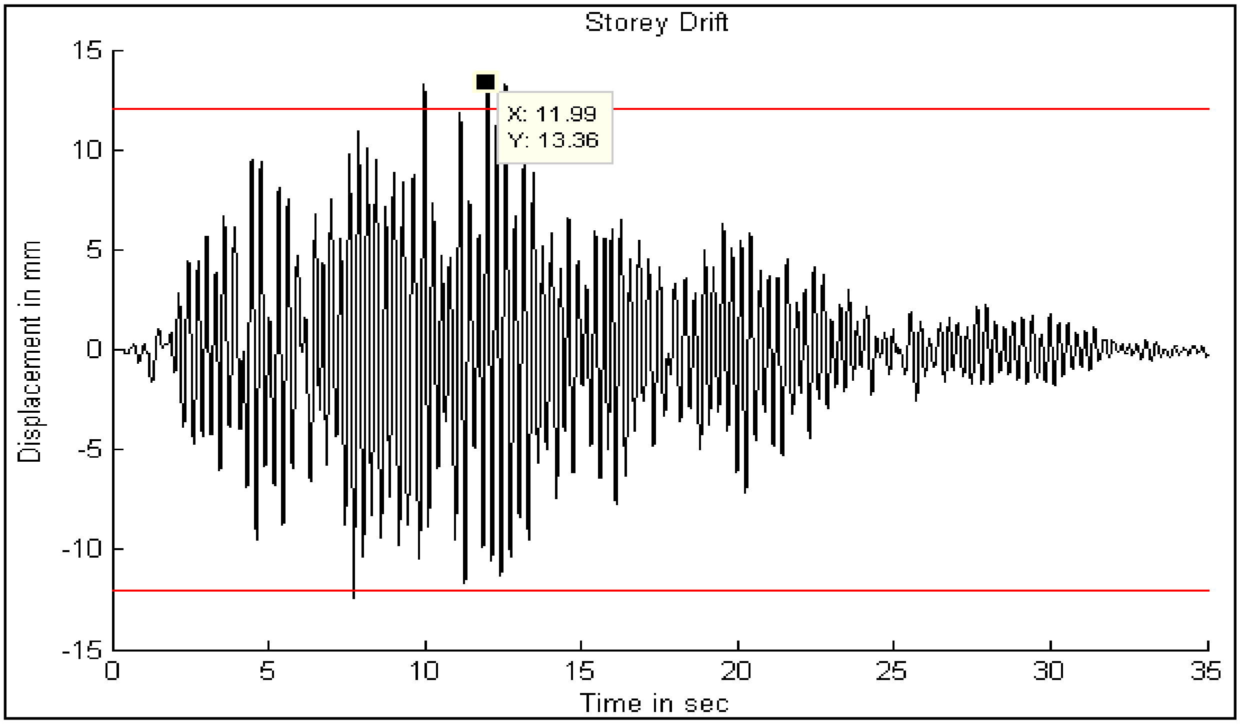


Fig. 4.13 Storey drift vs time response of the 2D concrete frame without floating column under IS code time history excitation

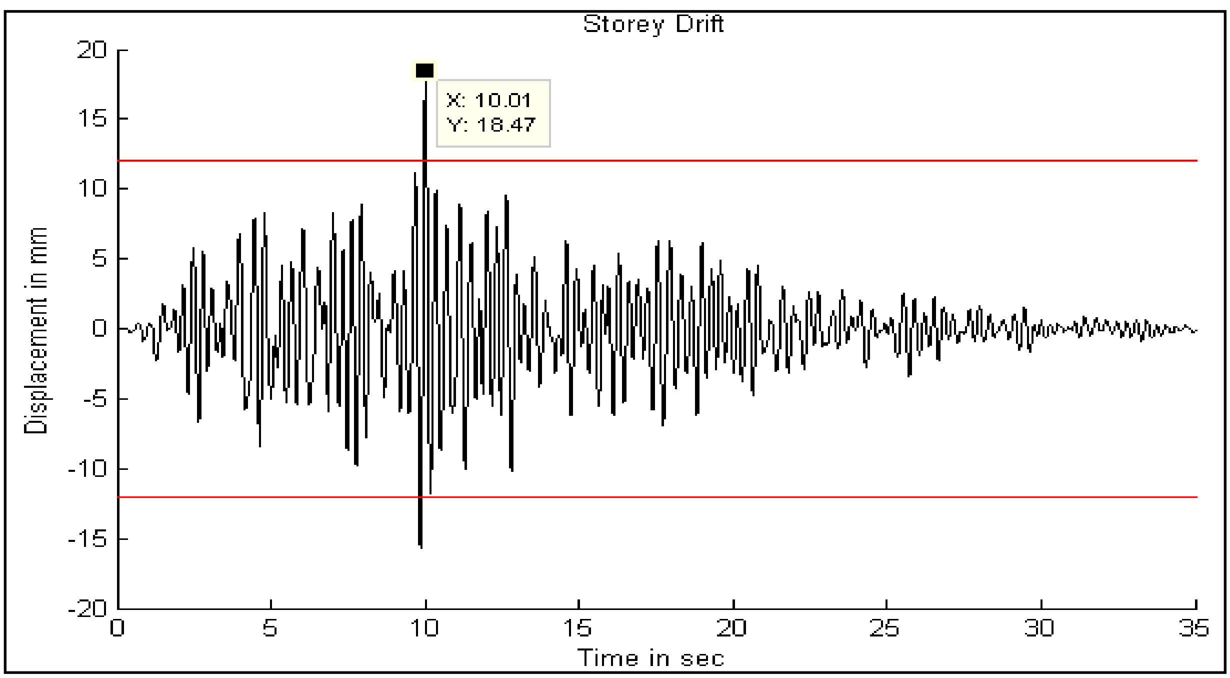


Fig. 4.14 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation

Table 4.10 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with and without floating column under IS code time history excitation

Maximum top floor displacement (mm)		% Increase
Frame with general columns	**Frame with floating column**	
12.61	17.14	35.92

Table 4.11 Comparison of predicted storey drift (mm) of the 2D concrete frame with and without floating column under IS code time history excitation

Storey drift (mm)			% Increase
Max storey drift as per IS Code (0.004h)	**Frame with general columns**	**Frame with floating column**	
12	13.36	18.47	38.25

Table 4.10 and 4.11 show that with the application of floating column in a frame the displacement and storey drift values are increasing abruptly. Hence the stiffness of the columns which are eventually transferring the load of the structure to the foundation are increased in further examples and responses are studied.

Example 4.6

In this example a concrete frame with floating column taken in Example 4.5 is analyzed by gradually increasing only the size of the ground floor column. The time history of top floor displacement is obtained and presented in figures 4.15-4.18. The maximum displacement of the top floor is obtained from the time history plot and tabulated in Table 4.12. It is observed that the maximum displacement decreases with strengthening the ground floor columns.

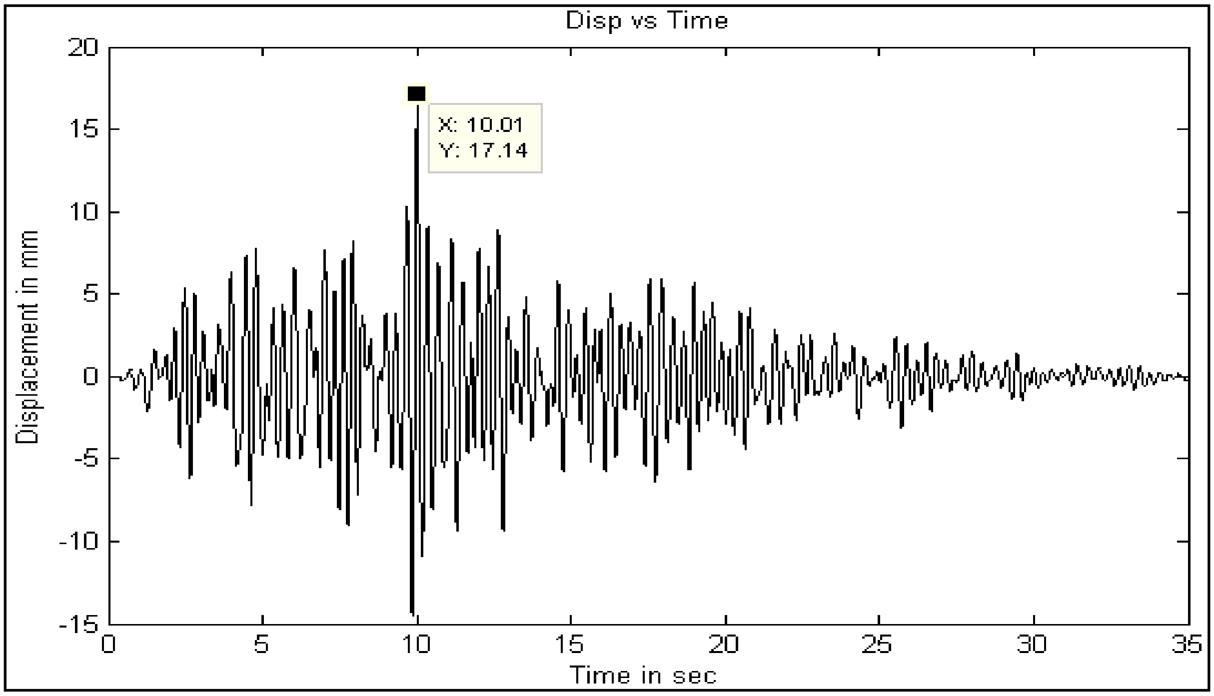


Fig. 4.15 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

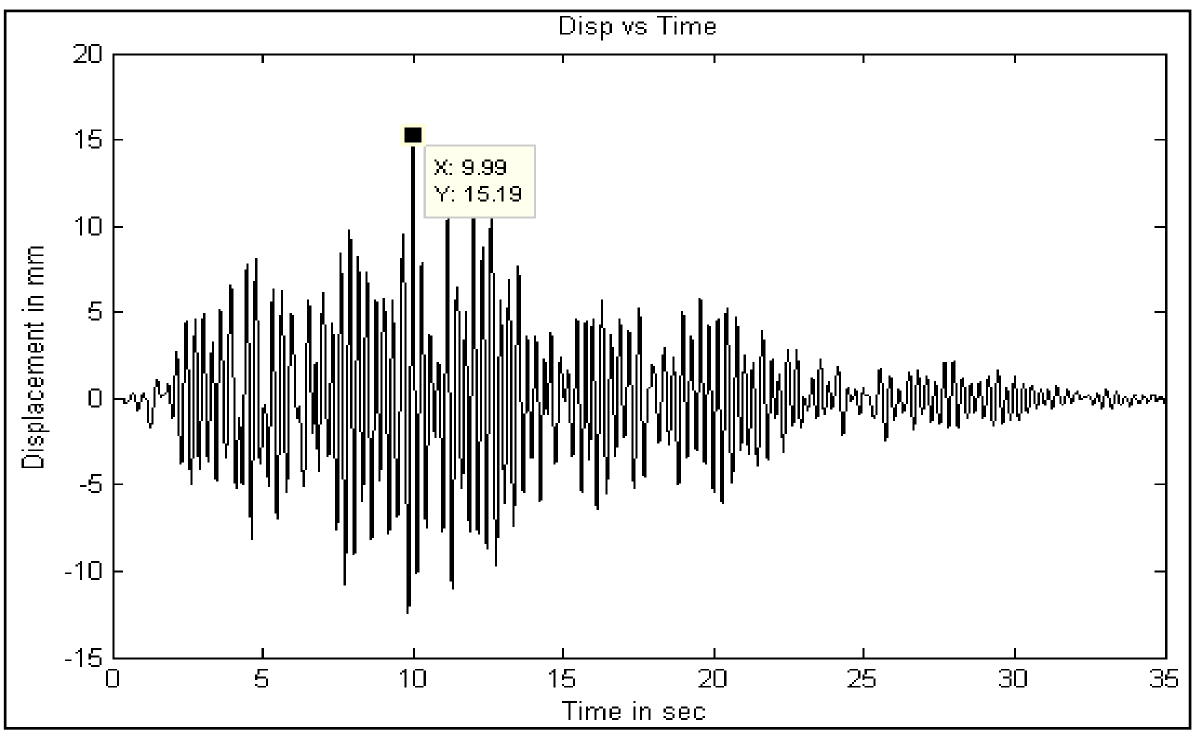


Fig. 4.16 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

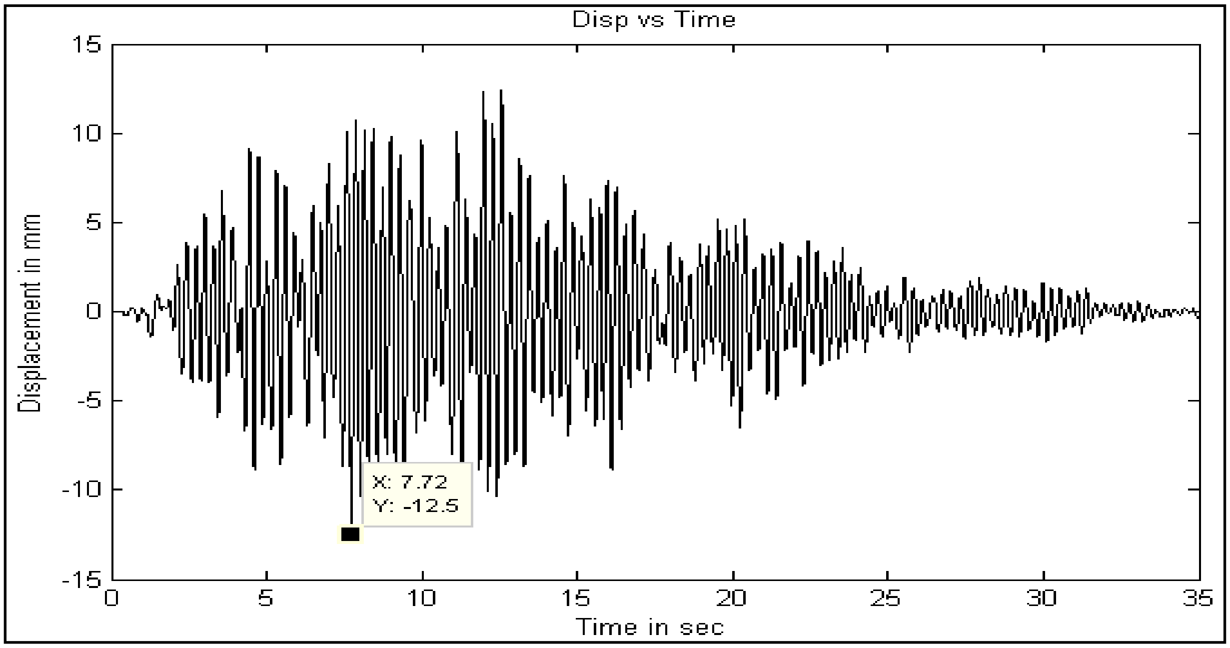


Fig. 4.17 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

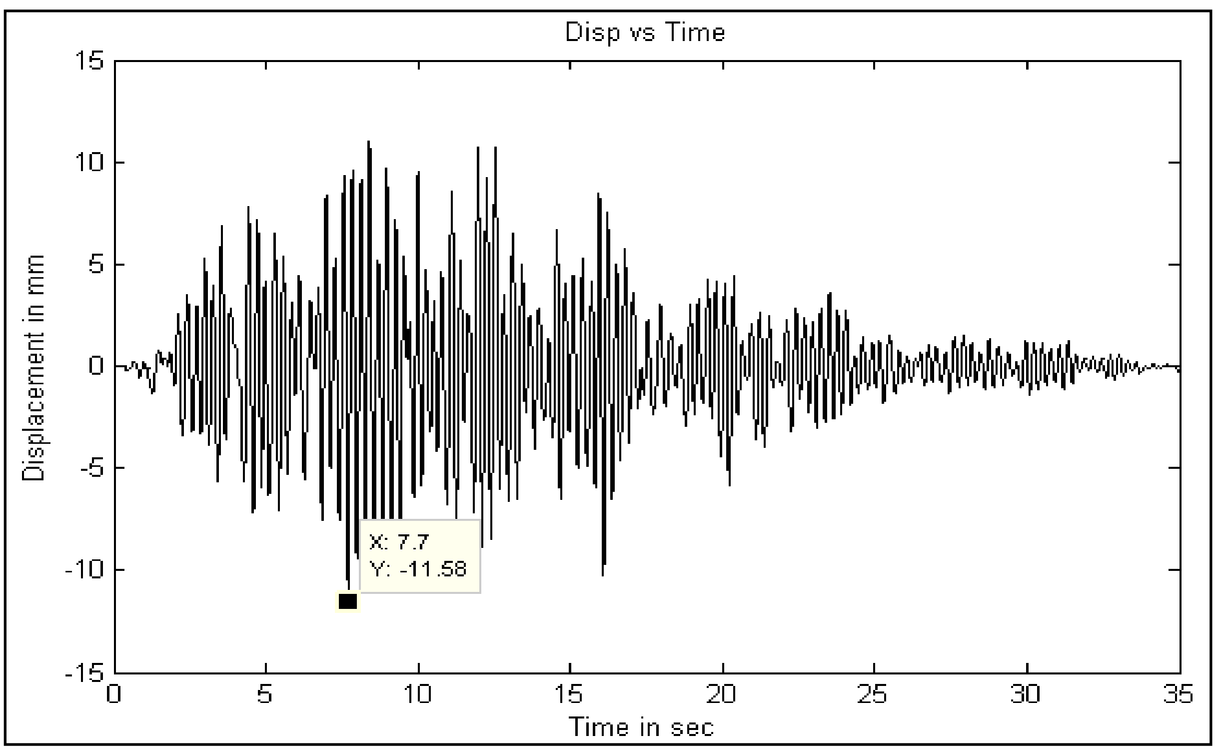


Fig. 4.18 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.12 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Max displacement (mm)	% Decrease
0.25 x 0.3	10.01	17.14	-
0.25 x 0.35	9.99	15.19	11.37
0.25 x 0.4	7.72	12.5	27.07
0.25 x 0.45	7.7	11.58	32.44

The time history of inter storey drift is obtained and presented in figures 4.19-4.22. The maximum inter storey drift is obtained from the time history plot and tabulated in Table 4.13. It is observed that the maximum inter storey drift decreases with strengthening the ground floor columns.

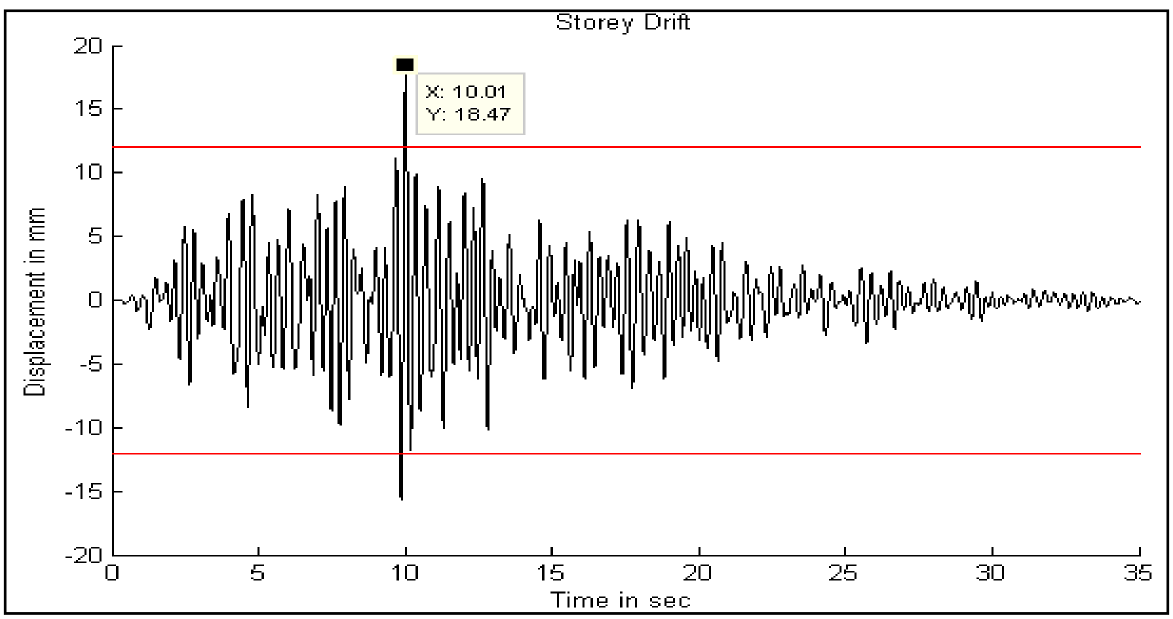


Fig. 4.19 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

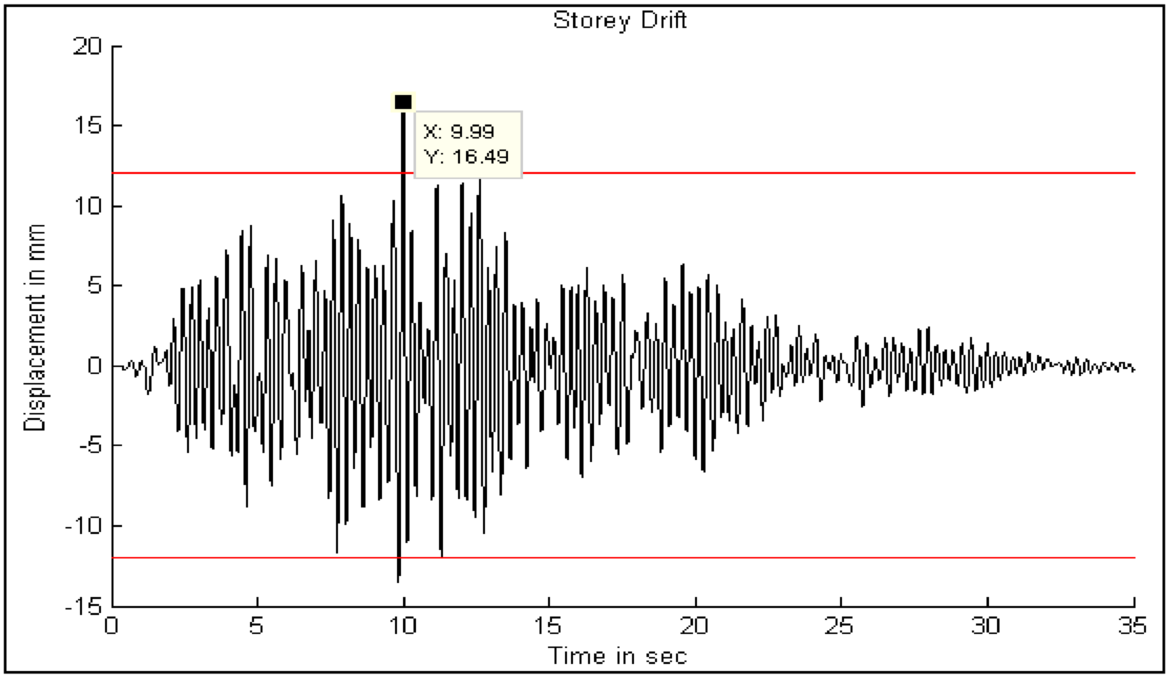


Fig. 4.20 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

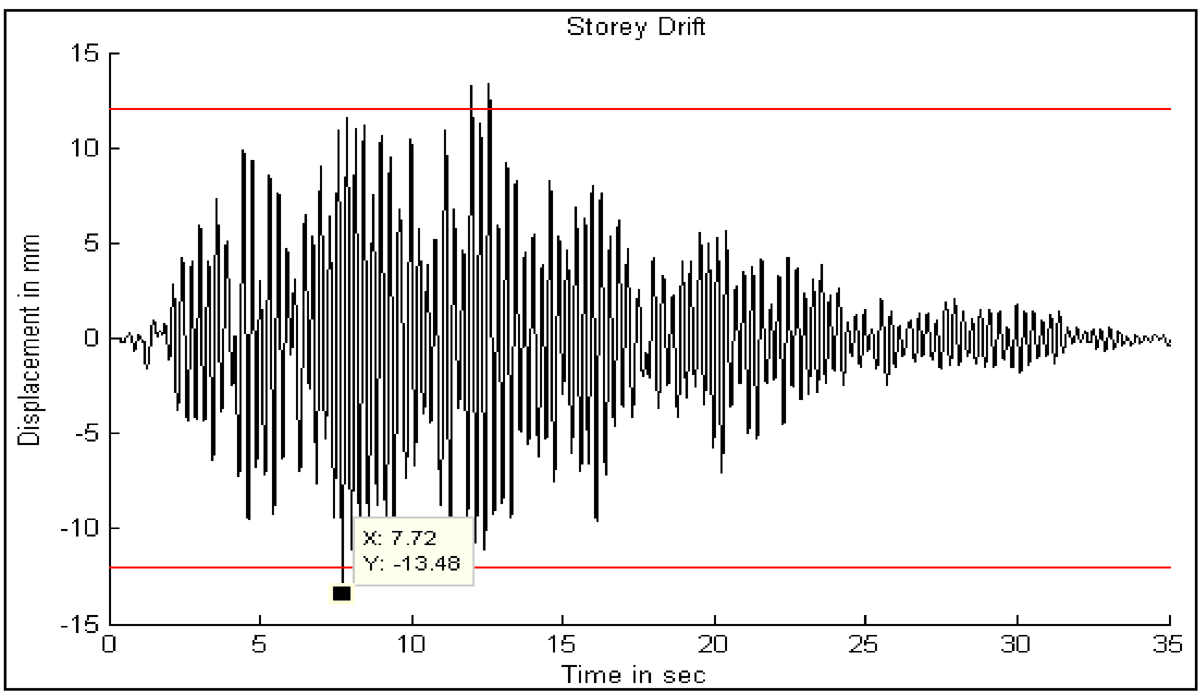


Fig. 4.21 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

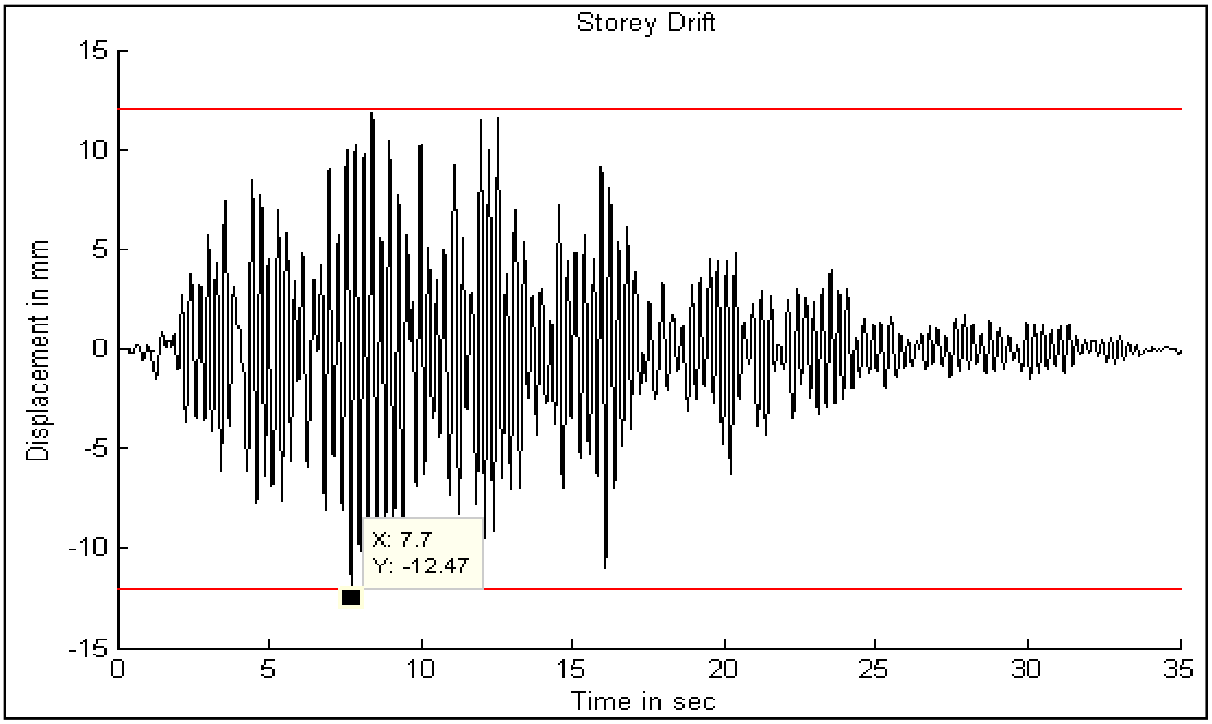


Fig. 4.22 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.13 Comparison of predicted storey drift (mm) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Storey drift (mm)	% Decrease
0.25 x 0.3	10.01	18.47	-
0.25 x 0.35	9.99	16.49	1072
0.25 x 0.4	7.72	13.48	27.02
0.25 x 0.45	7.7	12.47	32.48

The time history of base shear is obtained and presented in figures 4.23-4.26. The maximum base shear is obtained from the time history plot and tabulated in Table 4.14. It is observed that the maximum base shear decreases with strengthening the ground floor columns.

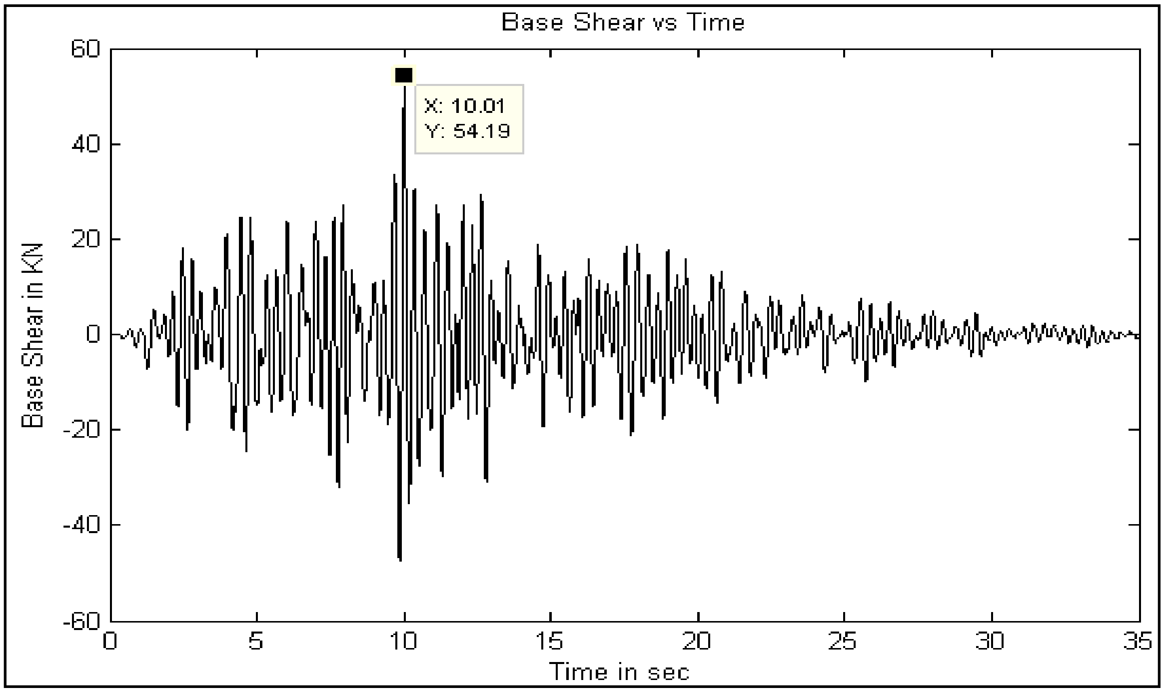


Fig. 4.23 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

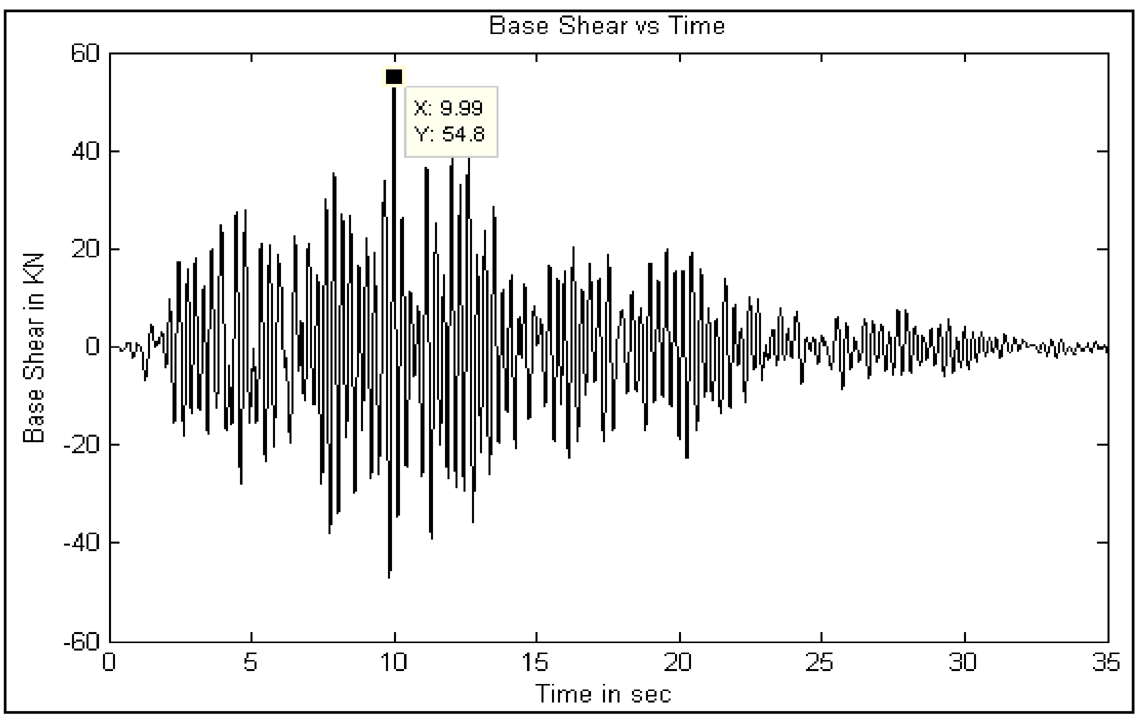


Fig. 4.24 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

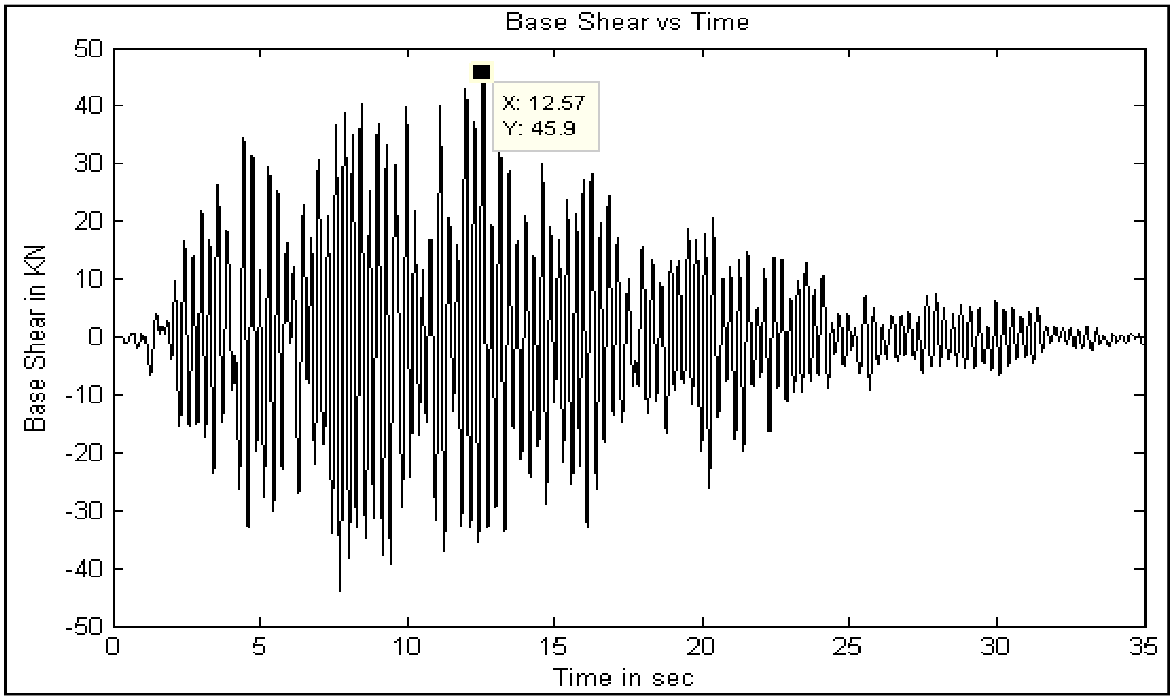


Fig. 4.25 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

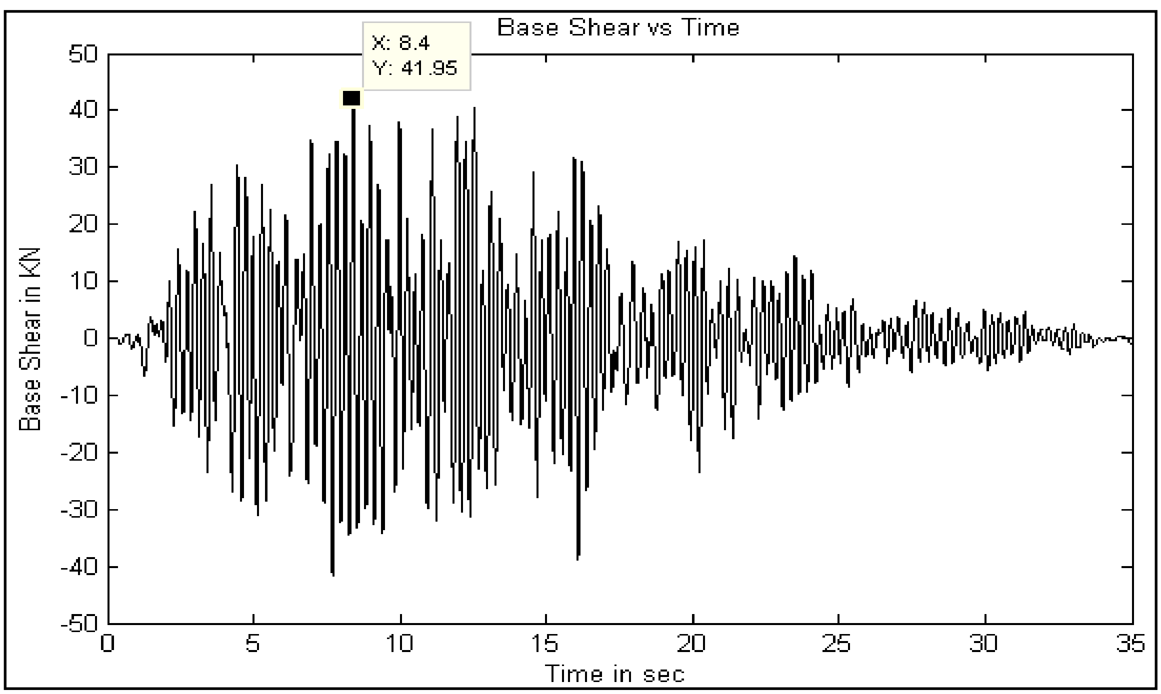


Fig. 4.26 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.14 Comparison of predicted base shear (kN) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Base shear (kN)	% Variation
0.25 x 0.3	10.01	54.19	-
0.25 x 0.35	9.99	54.8	1.12 (↑)
0.25 x 0.4	12.57	45.9	15.29 (↓)
0.25 x 0.45	8.4	41.95	22.58 (↓)

The time history of overturning moment is obtained and presented in figures 4.27-4.30. The maximum overturning moment is obtained from the time history plot and tabulated in Table 4.15. It is observed that the maximum overturning moment decreases with strengthening the ground floor columns.

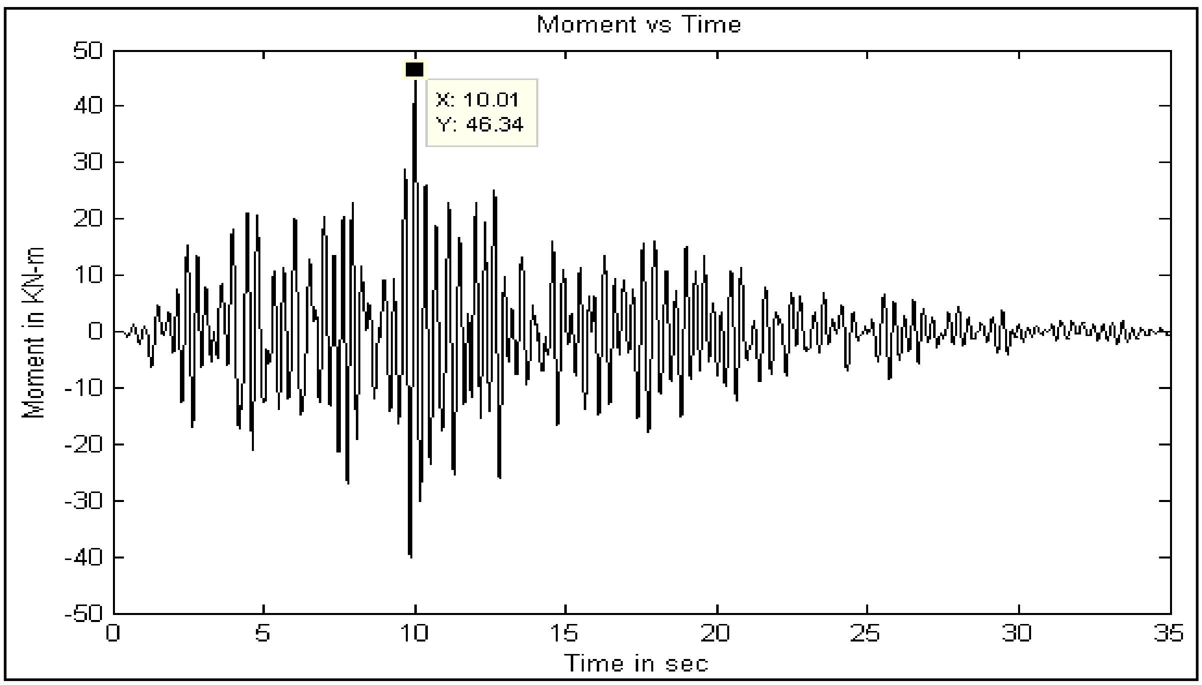


Fig. 4.27 Moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

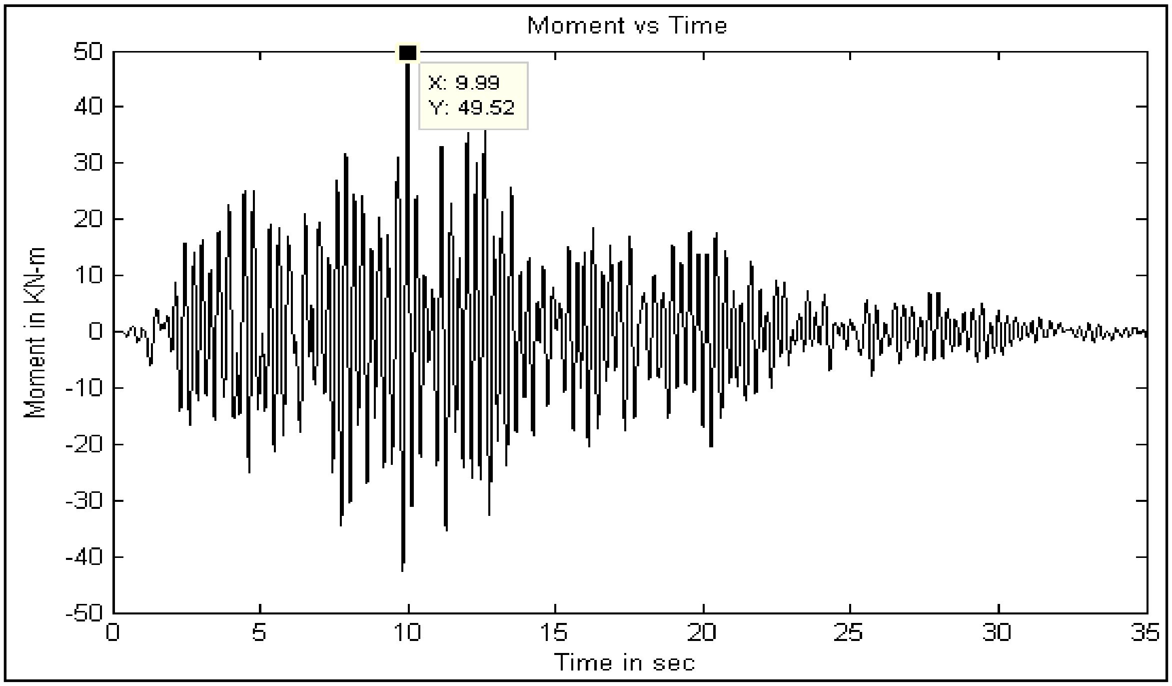


Fig. 4.28 Moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

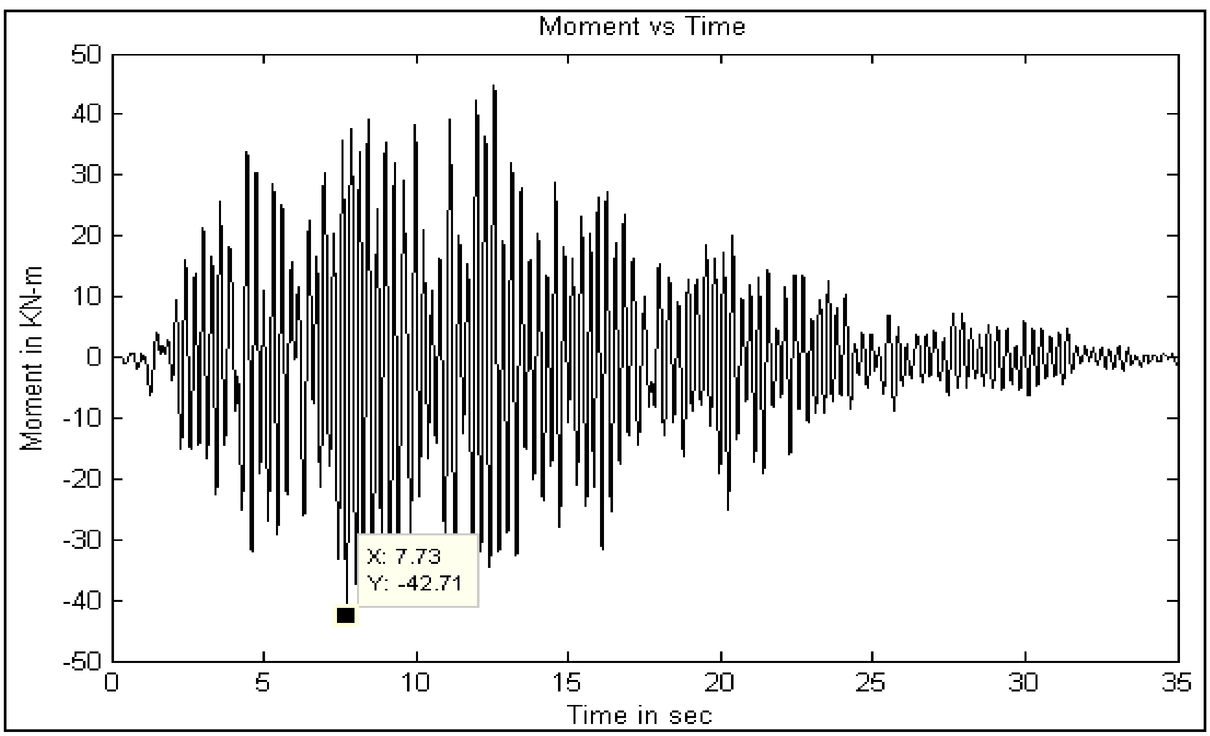


Fig. 4.29 Moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

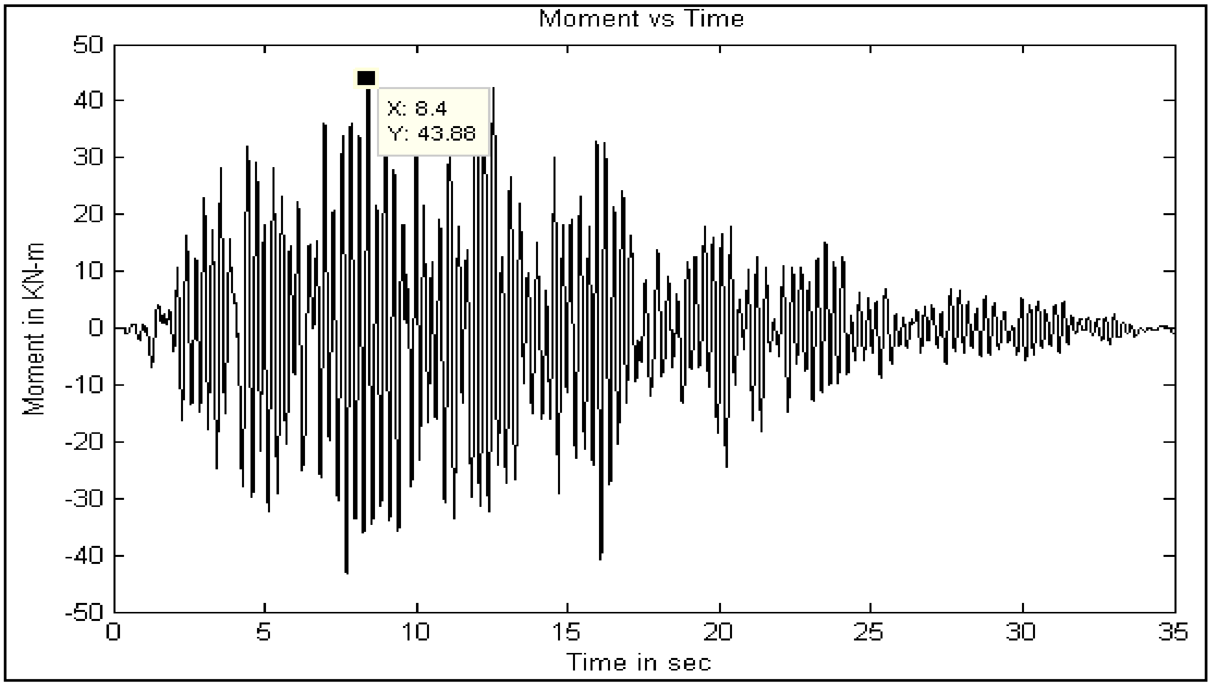


Fig. 4.30 Moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.15 Comparison of predicted maximum overturning moment (kN-m) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Maximum overturning moment (kN-m)	% Variation
0.25 x 0.3	10.01	46.34	-
0.25 x 0.35	9.99	49.52	6.86 (↑)
0.25 x 0.4	7.73	42.71	7.83 (↓)
0.25 x 0.45	8.4	43.88	5.31 (↓)

Example 4.7

In this example the same concrete frame with floating column taken in Example 4.5 is analyzed with size of both ground and first floor column in increasing order. The time history of maximum displacement is obtained and presented in figures 4.31-4.34. The maximum displacement is obtained from the time history plot and tabulated in Table 4.16. It is observed that the maximum displacement decreases with strengthening the ground floor columns.

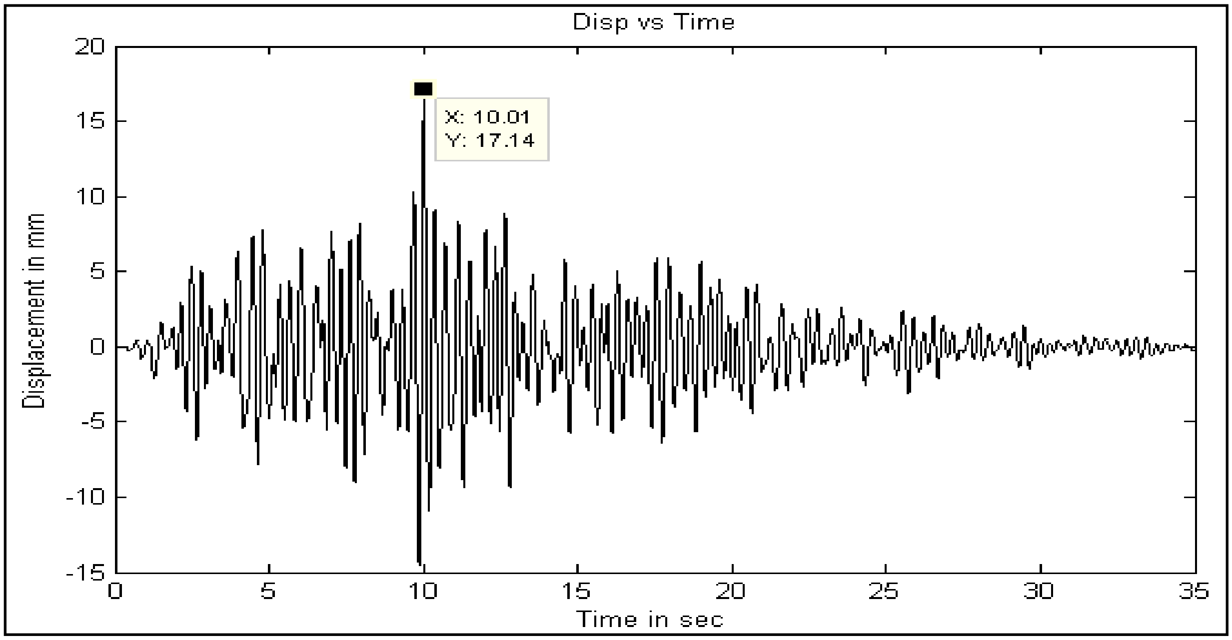


Fig. 4.31 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

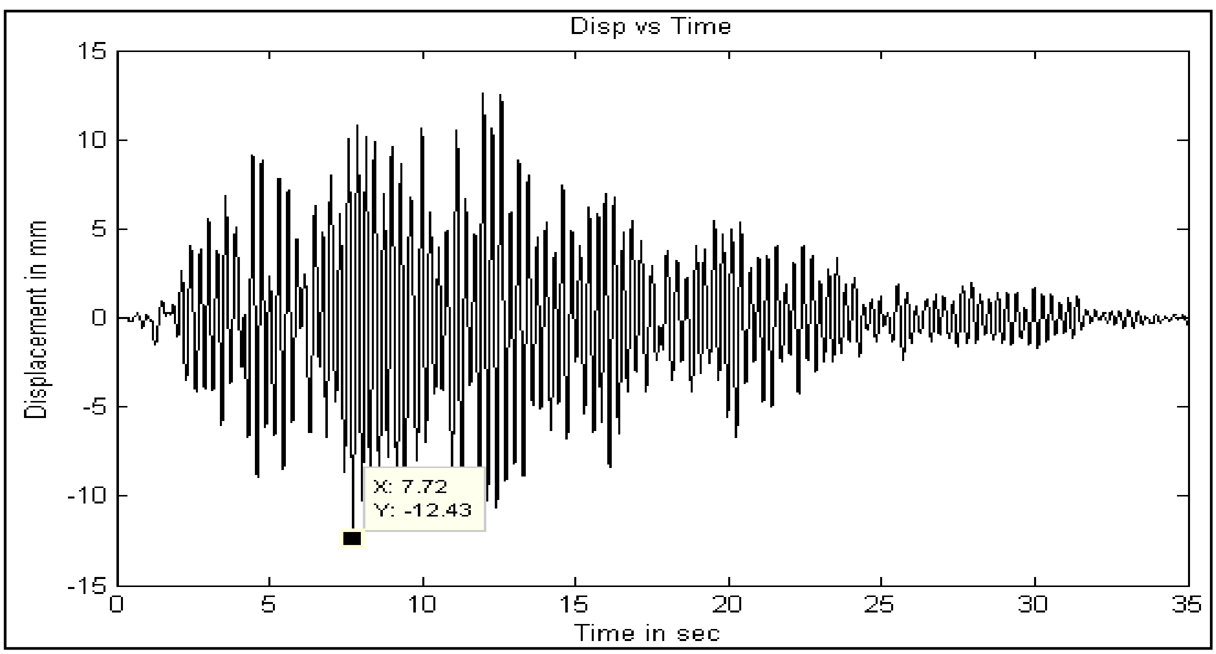


Fig. 4.32 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

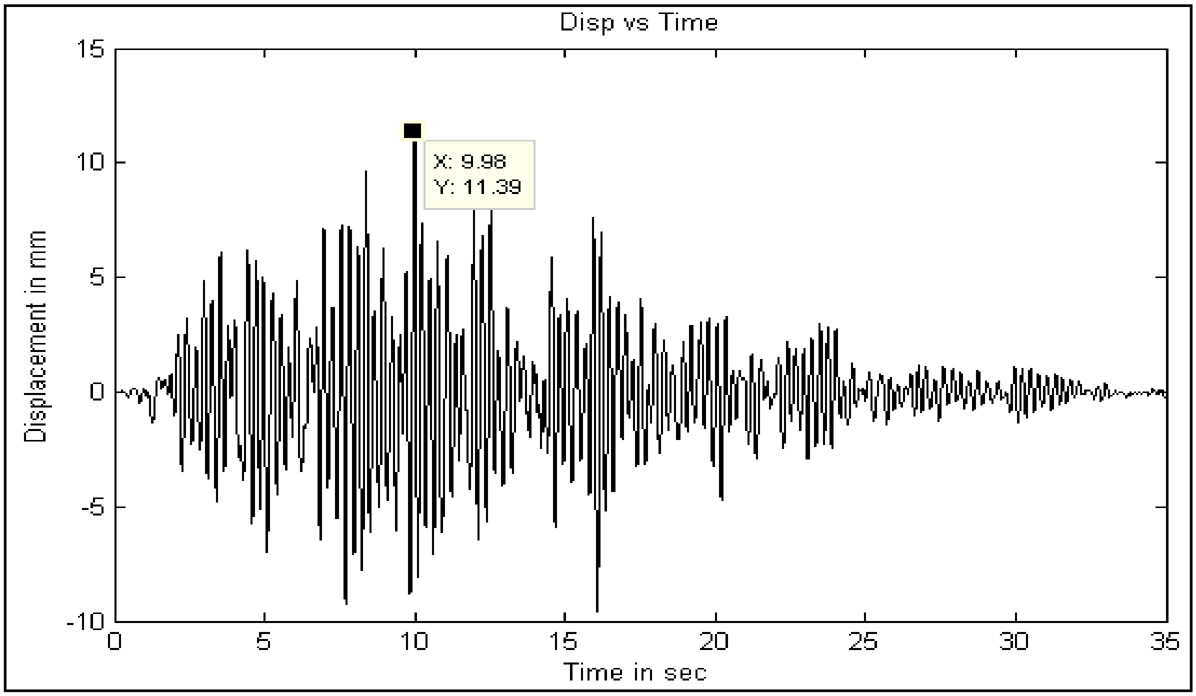


Fig. 4.33 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

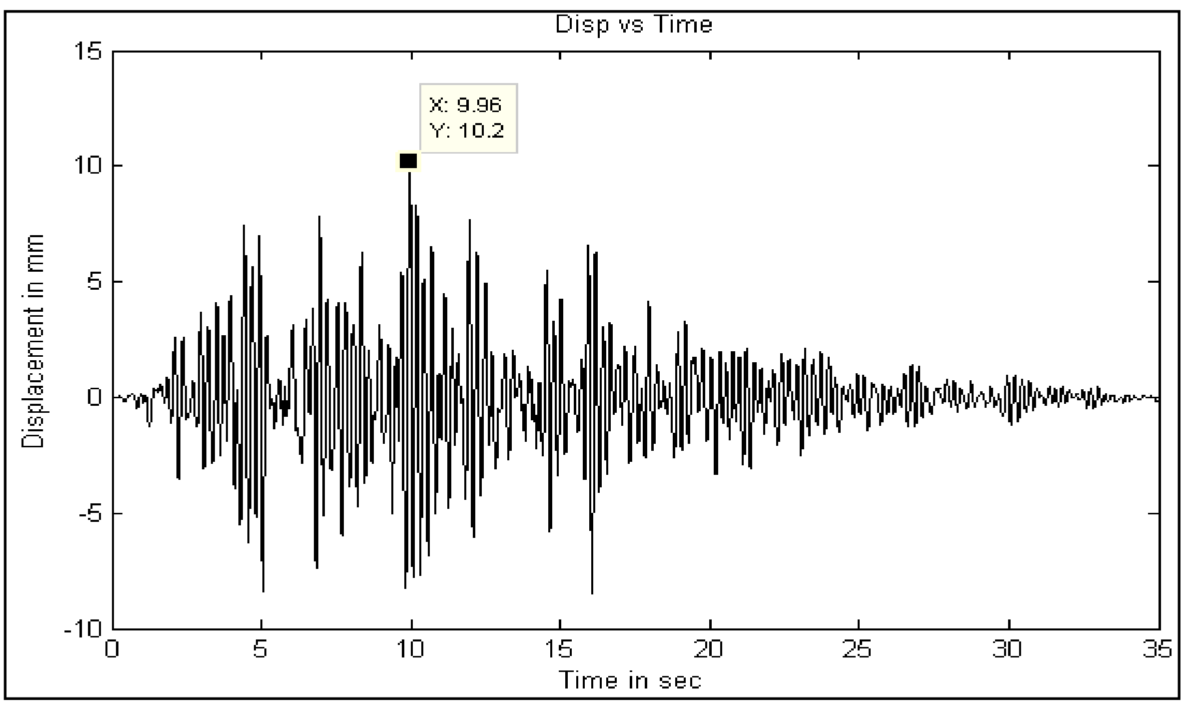


Fig. 4.34 Displacement vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.16 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Max displacement (mm)	% Decrease
0.25 x 0.3	10.01	17.14	-
0.25 x 0.35	7.72	12.43	27.48
0.25 x 0.4	9.98	11.39	33.55
0.25 x 0.45	9.96	10.2	40.49

The time history of inter storey drift is obtained and presented in figures 4.35-4.38. The maximum inter storey drift is obtained from the time history plot and tabulated in Table 4.17. It is observed that the maximum inter storey drift decreases with strengthening the ground floor columns.

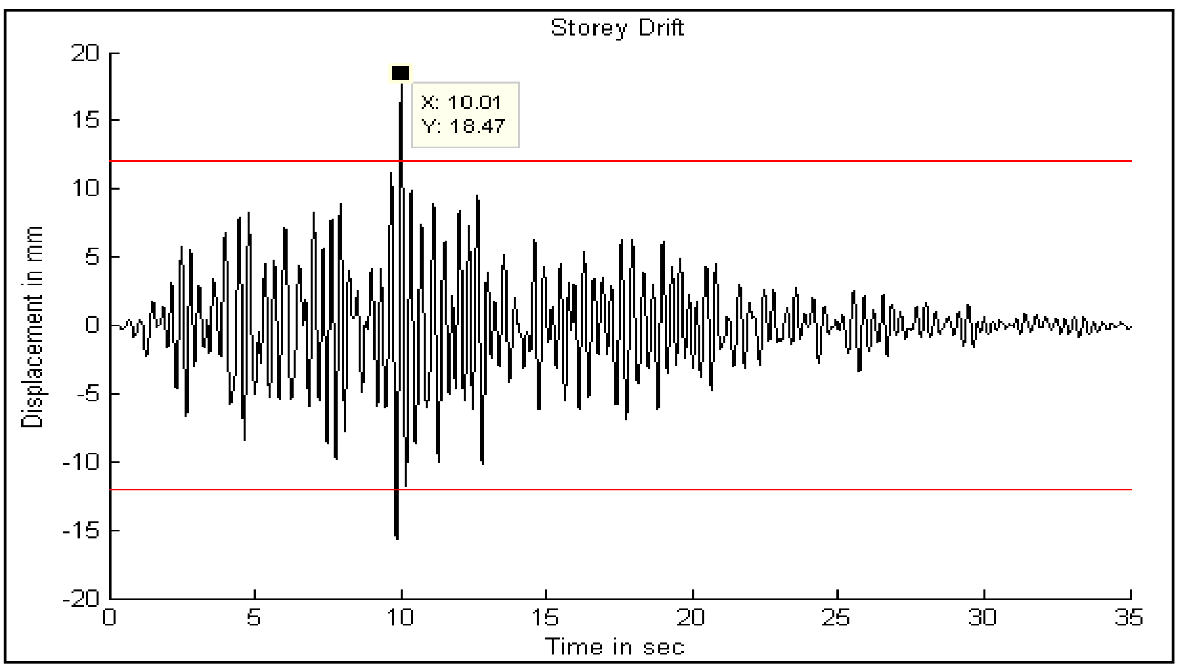


Fig. 4.35 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

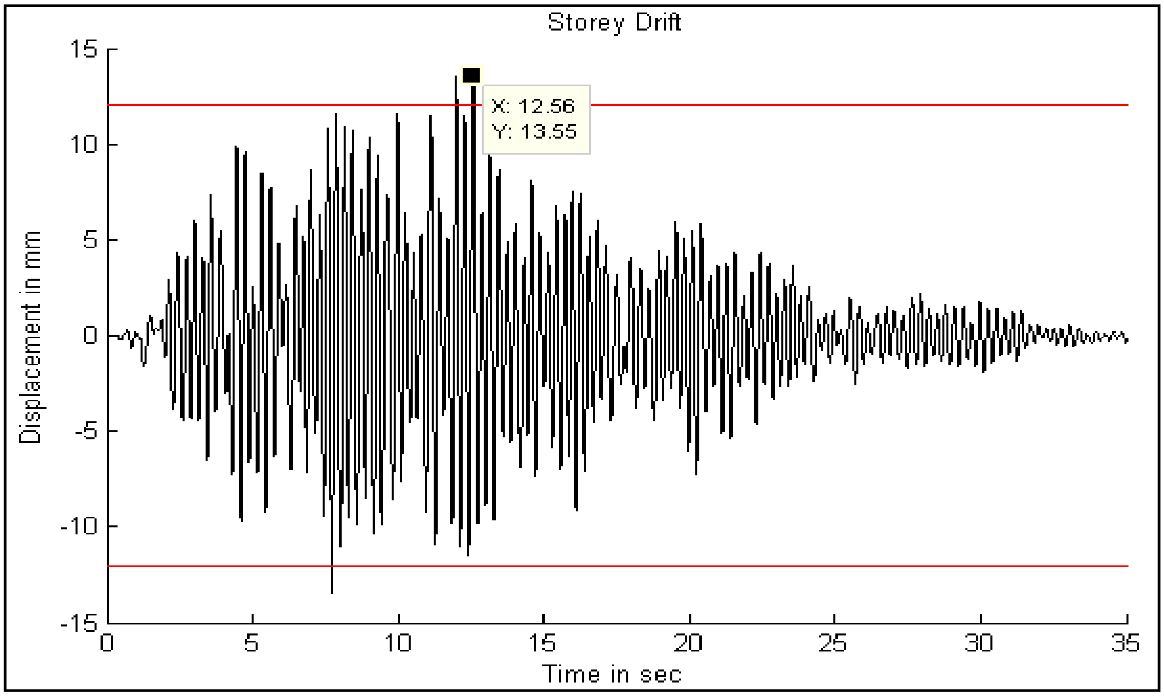


Fig. 4.36 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

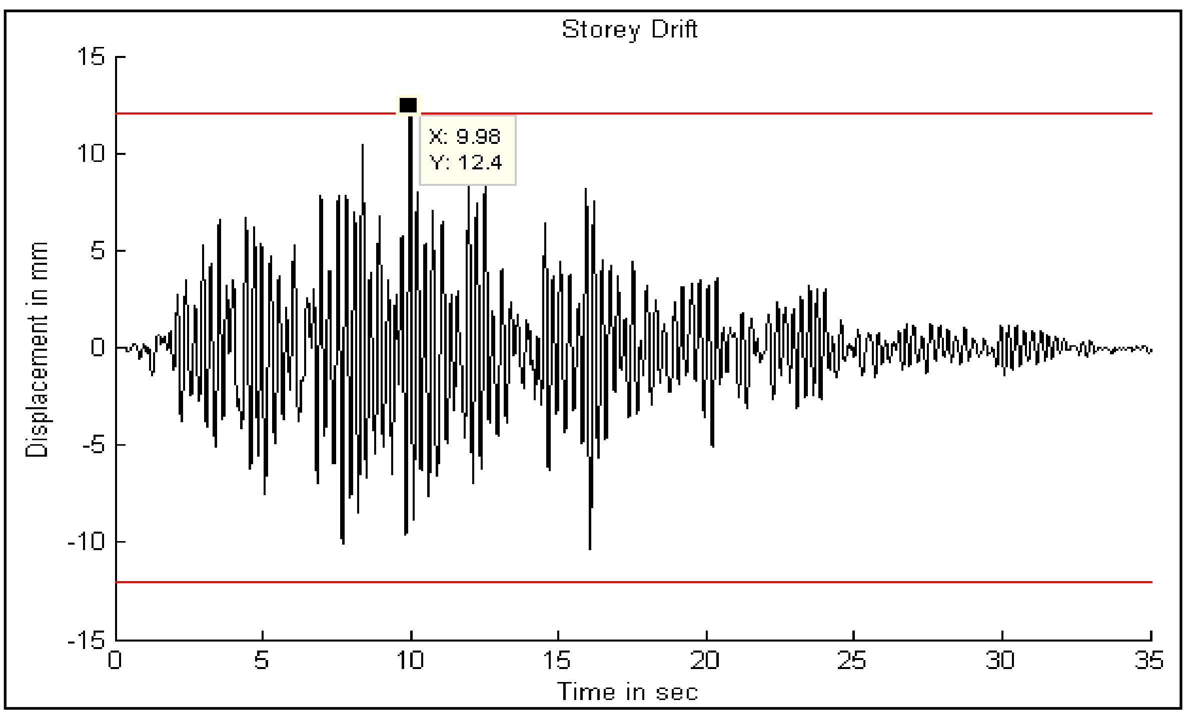


Fig. 4.37 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

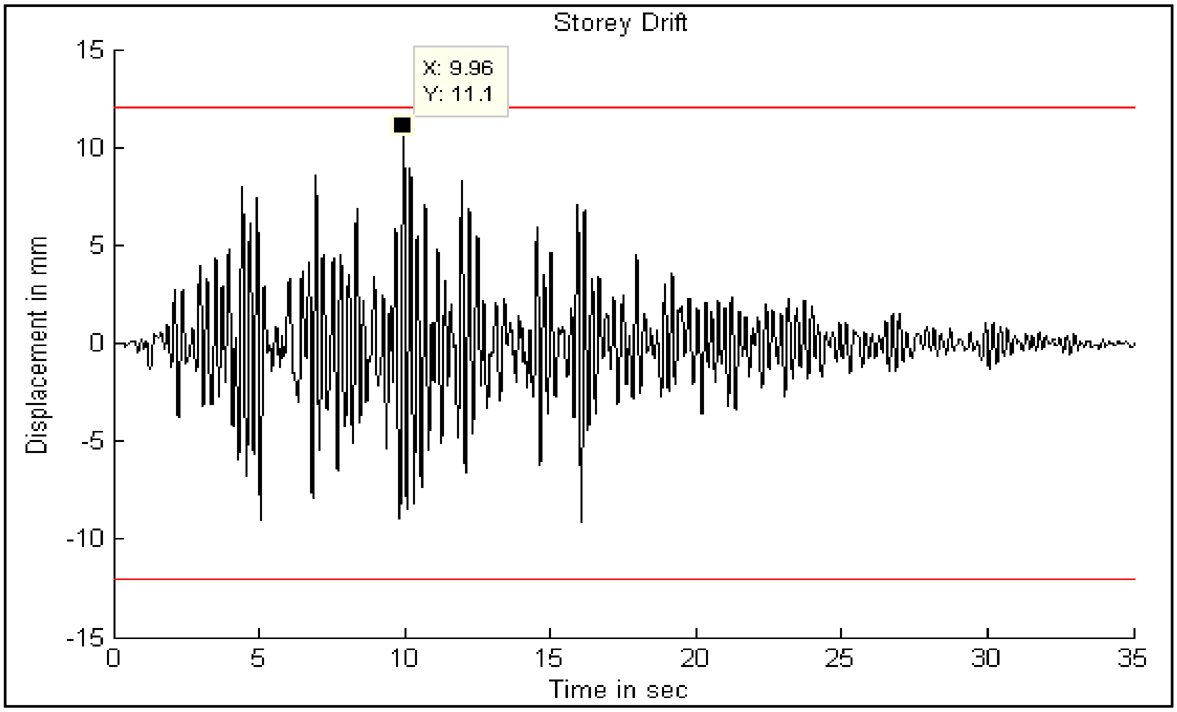


Fig. 4.38 Storey drift vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.17 Comparison of predicted maximum inter storey drift (mm) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum storey drift (mm)	% Decrease
0.25 x 0.3	10.01	18.47	-
0.25 x 0.35	12.56	13.55	26.64
0.25 x 0.4	9.98	12.4	32.86
0.25 x 0.45	9.96	11.1	39.9

The time history of base shear is obtained and presented in figures 4.39-4.42. The maximum base shear is obtained from the time history plot and tabulated in Table 4.18. It is observed that the maximum base shear increases with strengthening the ground floor columns.

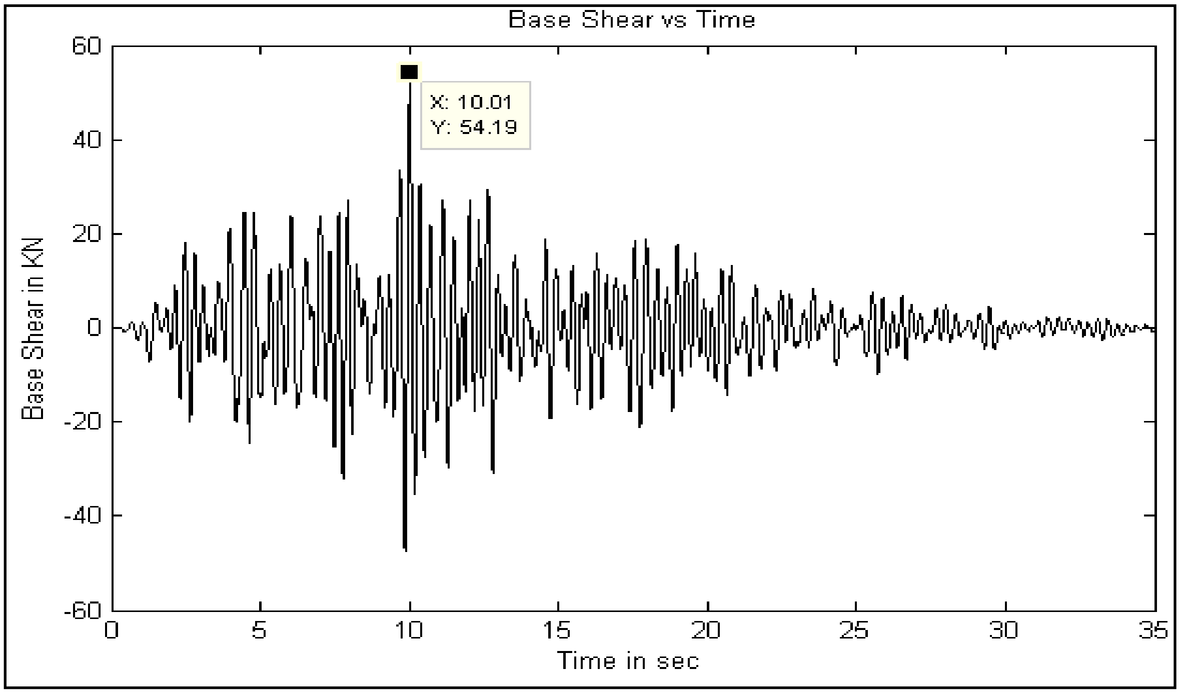


Fig. 4.39 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

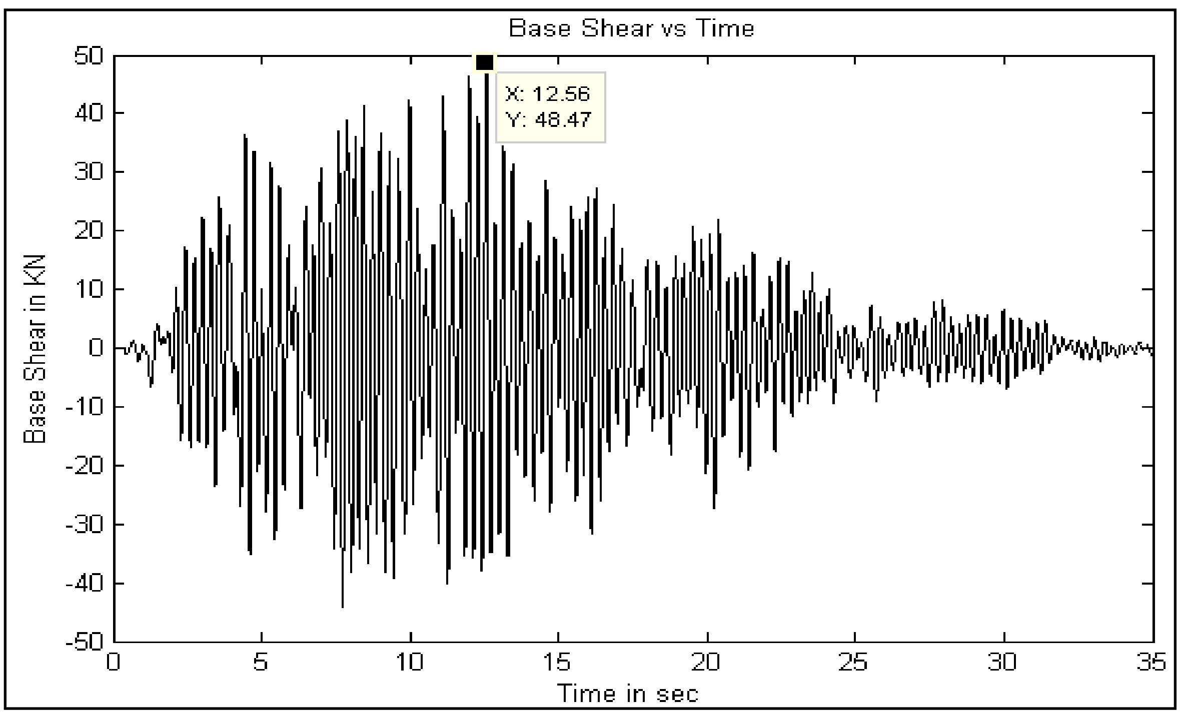


Fig. 4.40 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

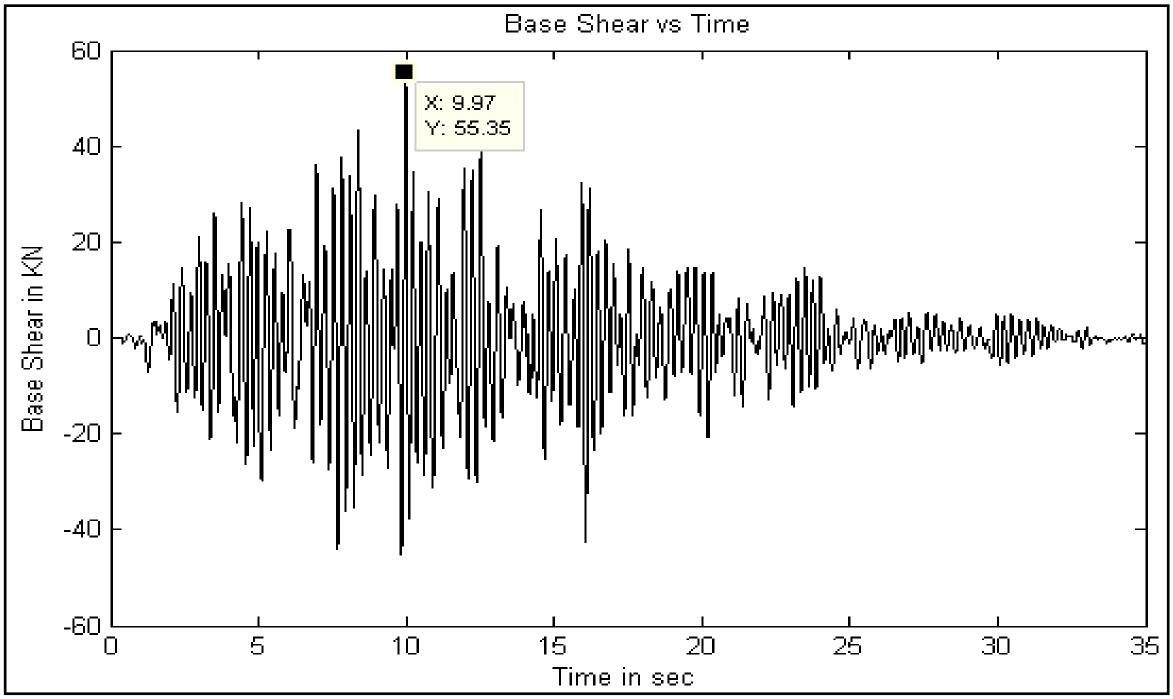


Fig. 4.41 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

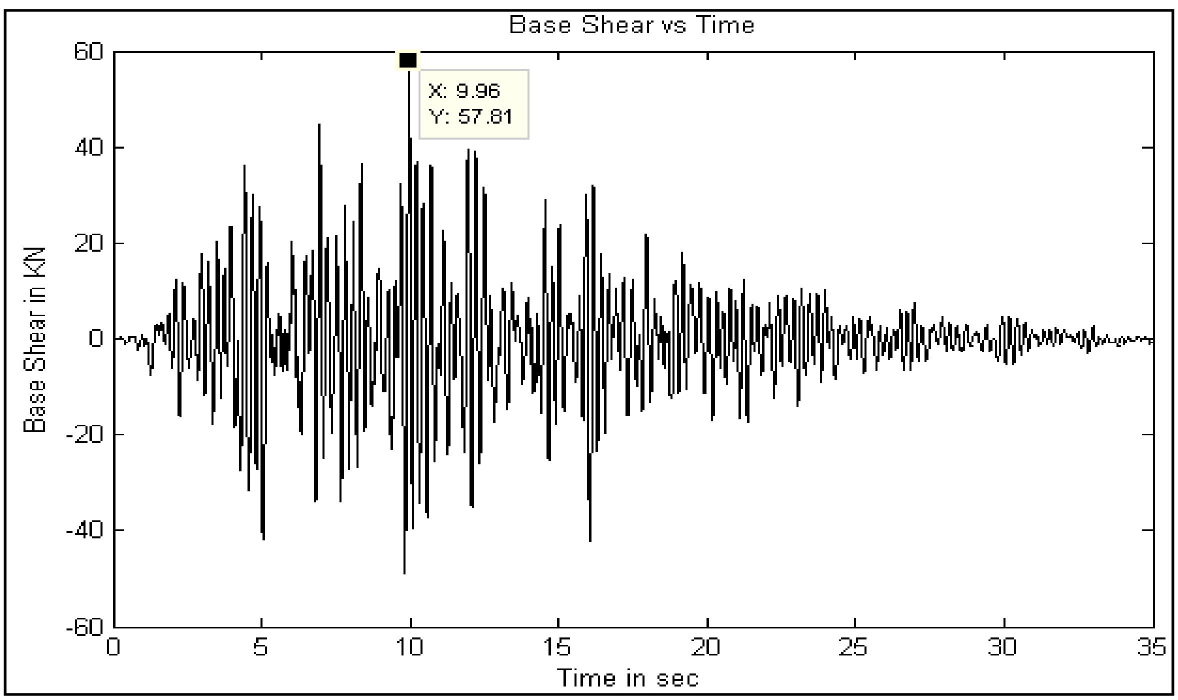


Fig. 4.42 Base shear vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.18 Comparison of predicted maximum base shear (kN) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum base shear (kN)	% Variation
0.25 x 0.3	10.01	54.19	-
0.25 x 0.35	12.56	48.47	10.55 (↓)
0.25 x 0.4	9.97	55.35	2.14 (↑)
0.25 x 0.45	9.96	57.81	6.68 (↑)

The time history of overturning moment is obtained and presented in figures 4.43-4.46. The maximum overturning moment is obtained from the time history plot and tabulated in Table 4.19. It is observed that the maximum overturning moment increases with strengthening the ground floor columns.

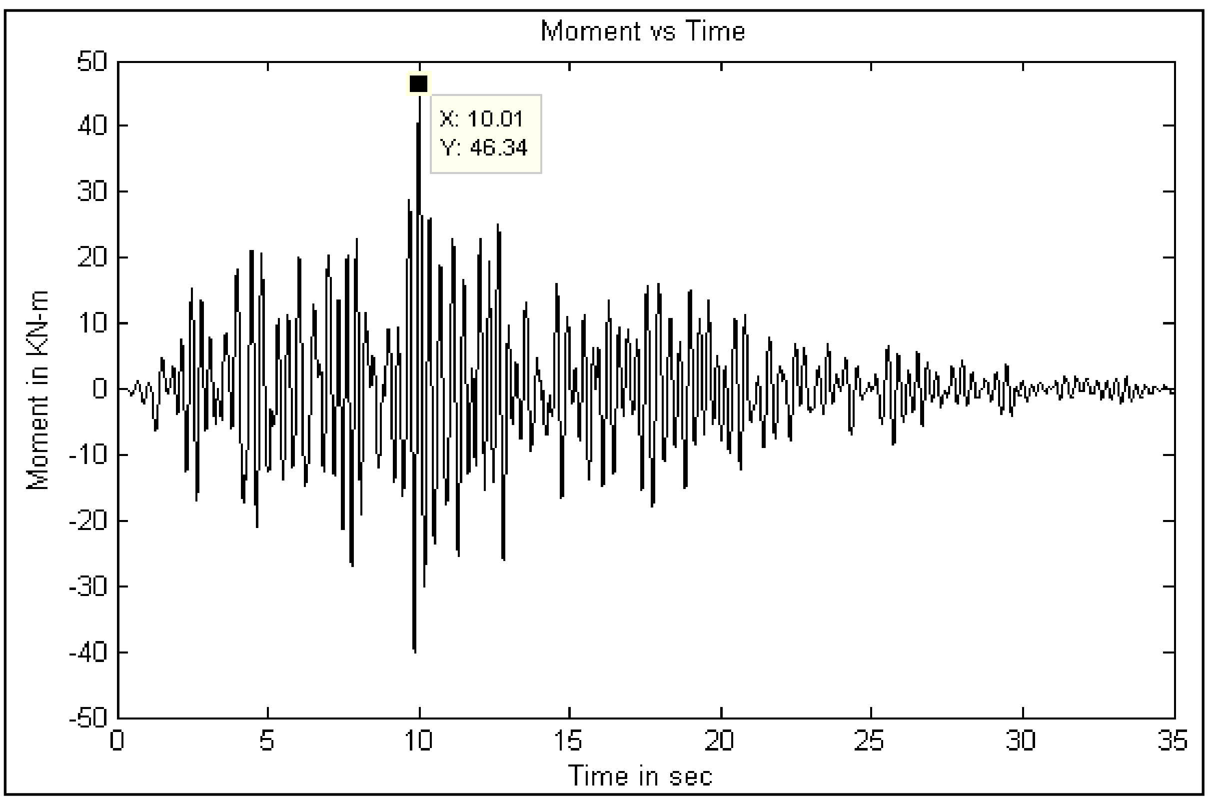


Fig. 4.43 Overturning moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.3 m)

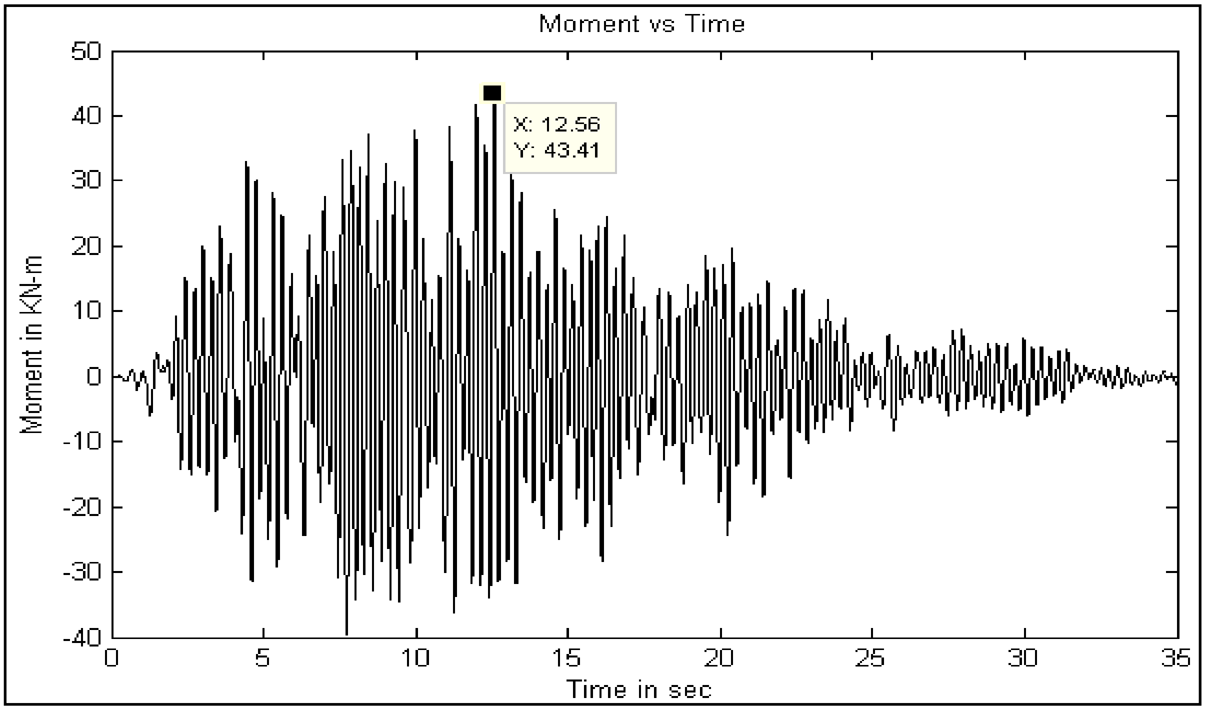


Fig. 4.44 Overturning moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.35 m)

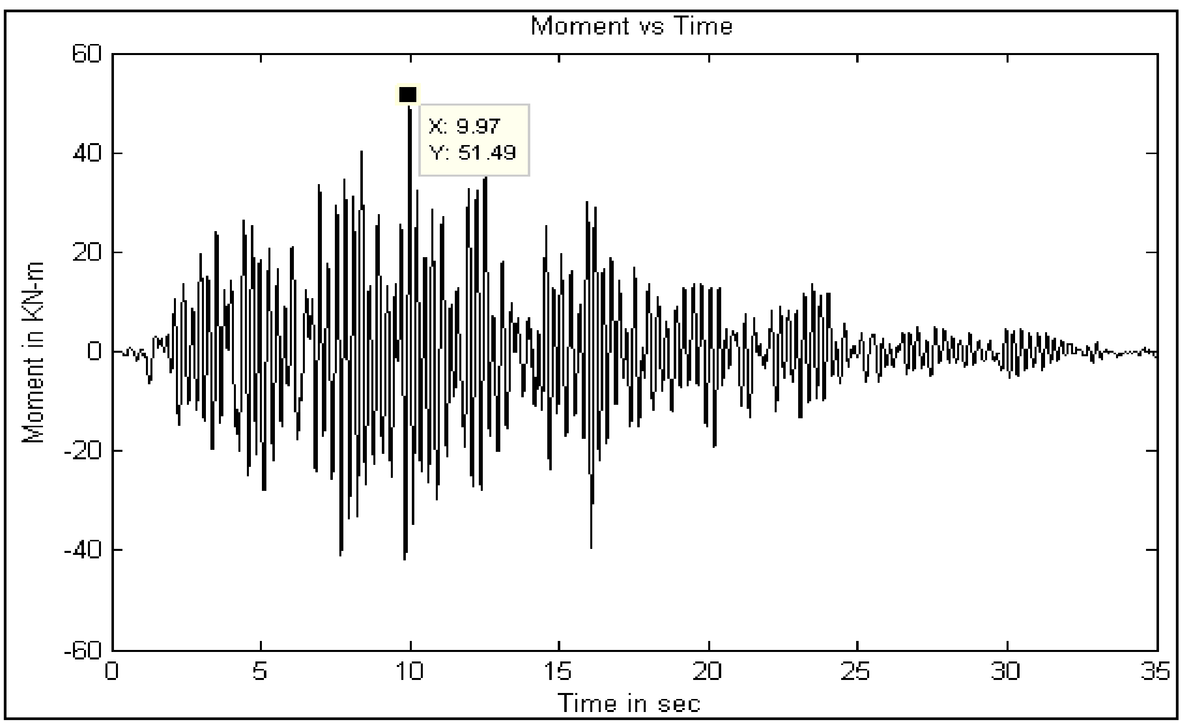


Fig. 4.45 Overturning moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.4 m)

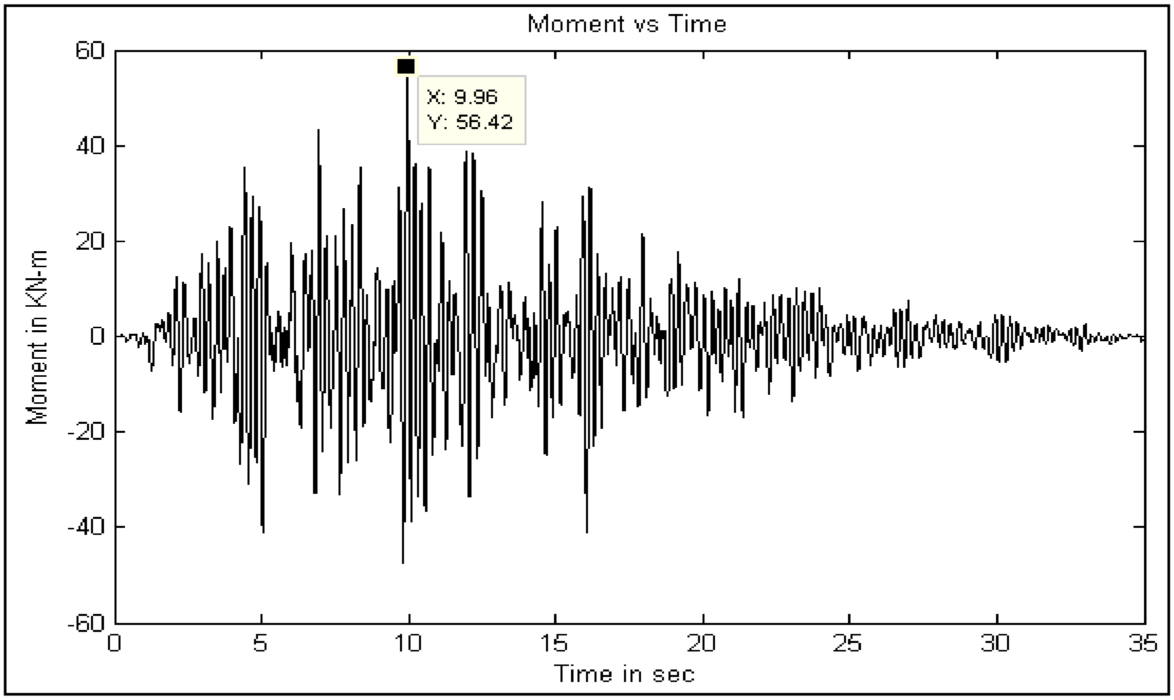


Fig. 4.46 Overturning moment vs time response of the 2D concrete frame with floating column under IS code time history excitation (Column size- 0.25 x 0.45 m)

Table 4.19 Comparison of predicted maximum overturning moment (kN-m) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum overturning moment (kN-m)	% Variation
0.25 x 0.3	10.01	46.34	-
0.25 x 0.35	12.56	43.41	6.32 (↓)
0.25 x 0.4	9.97	51.49	11.11 (↑)
0.25 x 0.45	9.96	56.42	21.75 (↑)

Example 4.8

In this example the same problem in Example 4.6 is analyzed under Elcentro(EW) earthquake time history data. Elcentro time history data is a low frequency content data. It has PGA of 0.2141g. But to compare the response of the structure under IS code time history data it is also scaled down to 0.2g. The duration of excitation is also taken upto 40sec.

The time history of displacement is obtained and presented in figures 4.47-4.50. The maximum displacement is obtained from the time history plot and tabulated in Table 4.20. It is observed that the maximum displacement decreases with strengthening the ground floor columns.

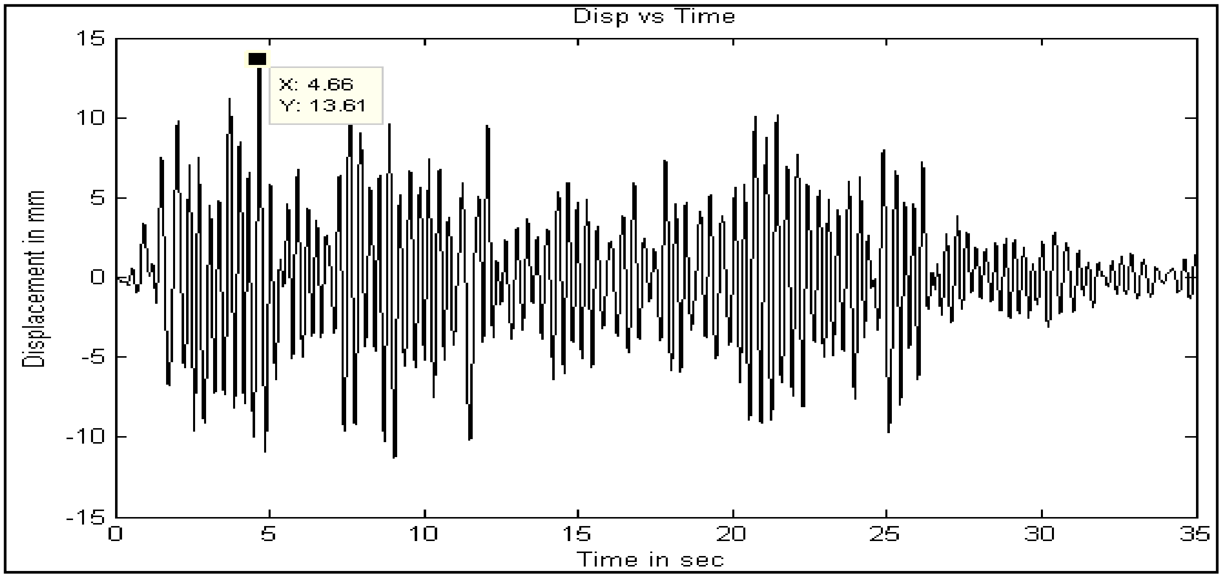


Fig. 4.47 Displacement vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

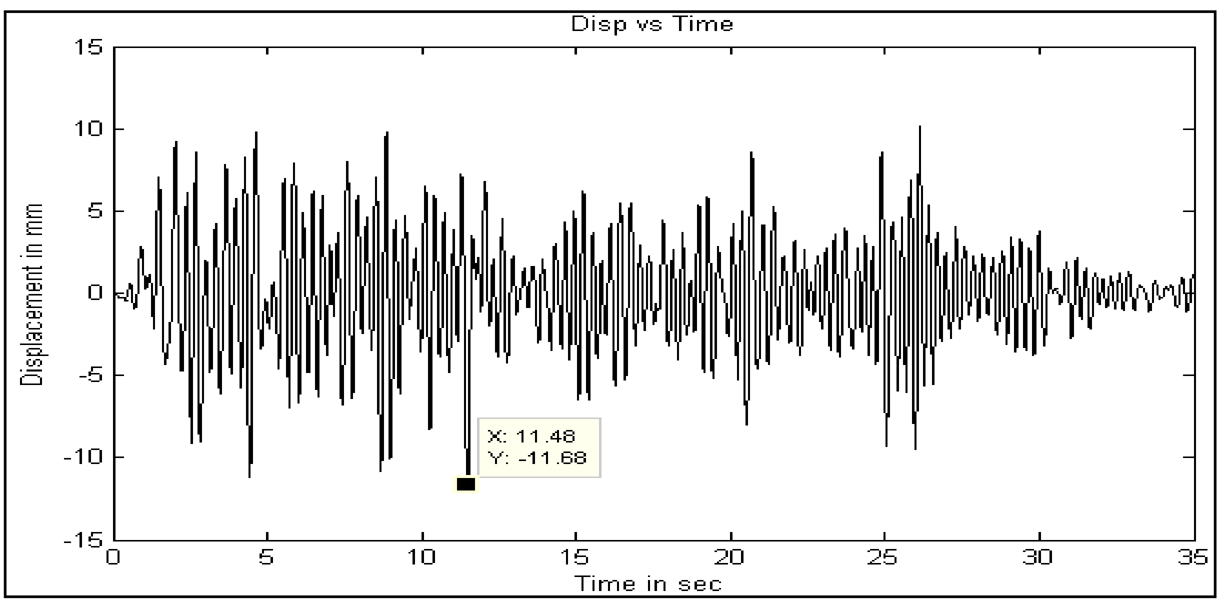


Fig. 4.48 Displacement vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

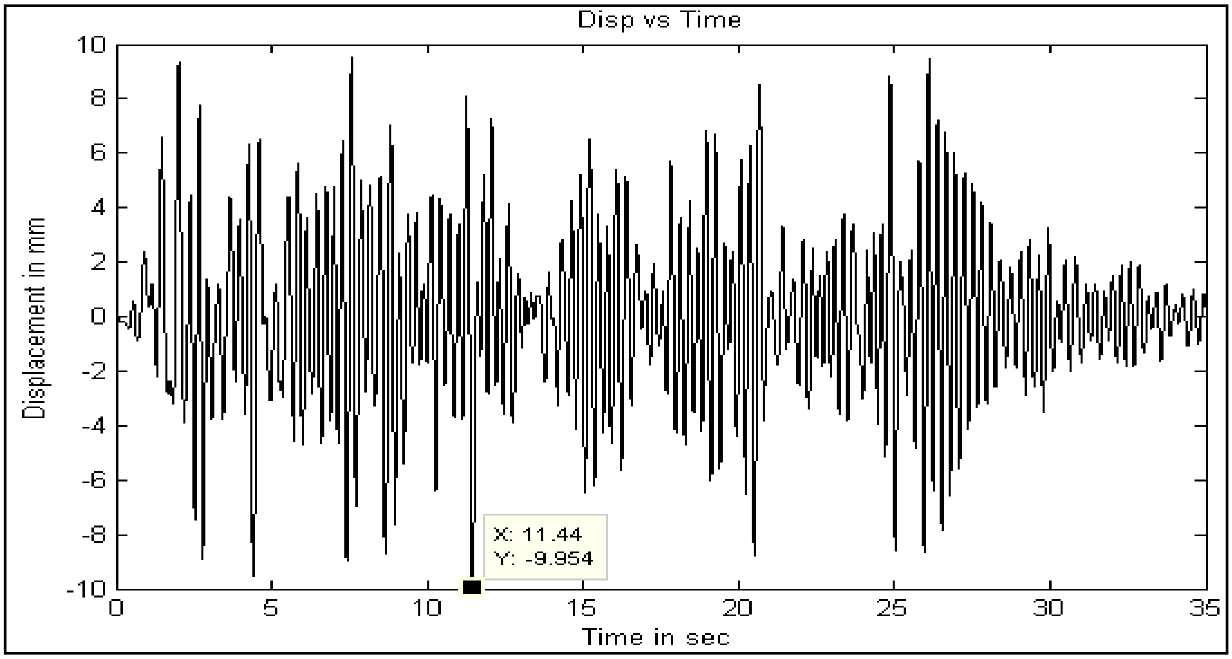


Fig. 4.49 Displacement vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.4 m)

Table 4.20 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Max displacement (mm)	% Decrease
0.25 x 0.3	4.66	13.61	-
0.25 x 0.35	11.48	11.68	14.18
0.25 x 0.4	11.44	9.954	26.86

The time history of inter storey drift is obtained and presented in figures 4.50-4.52. The maximum inter storey drift is obtained from the time history plot and tabulated in Table 4.21. It is observed that the maximum inter storey drift decreases with strengthening the ground floor columns.

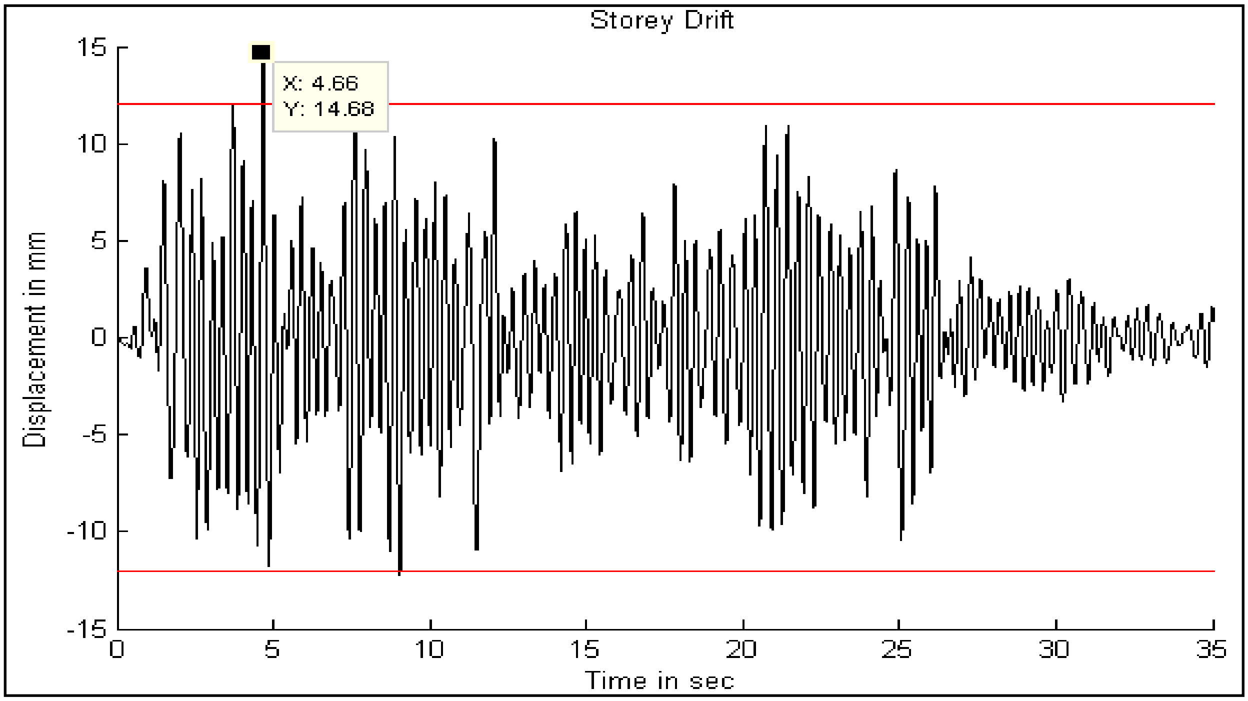


Fig. 4.50 Storey drift vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

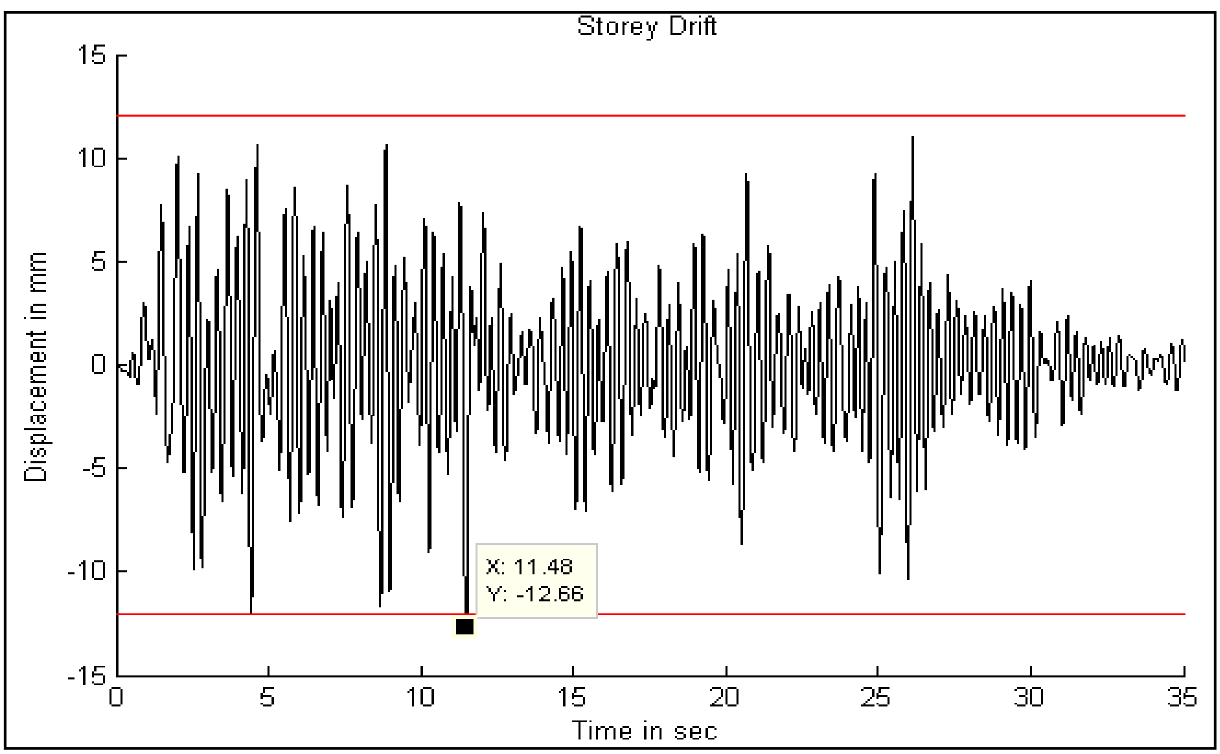


Fig. 4.51 Storey drift vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

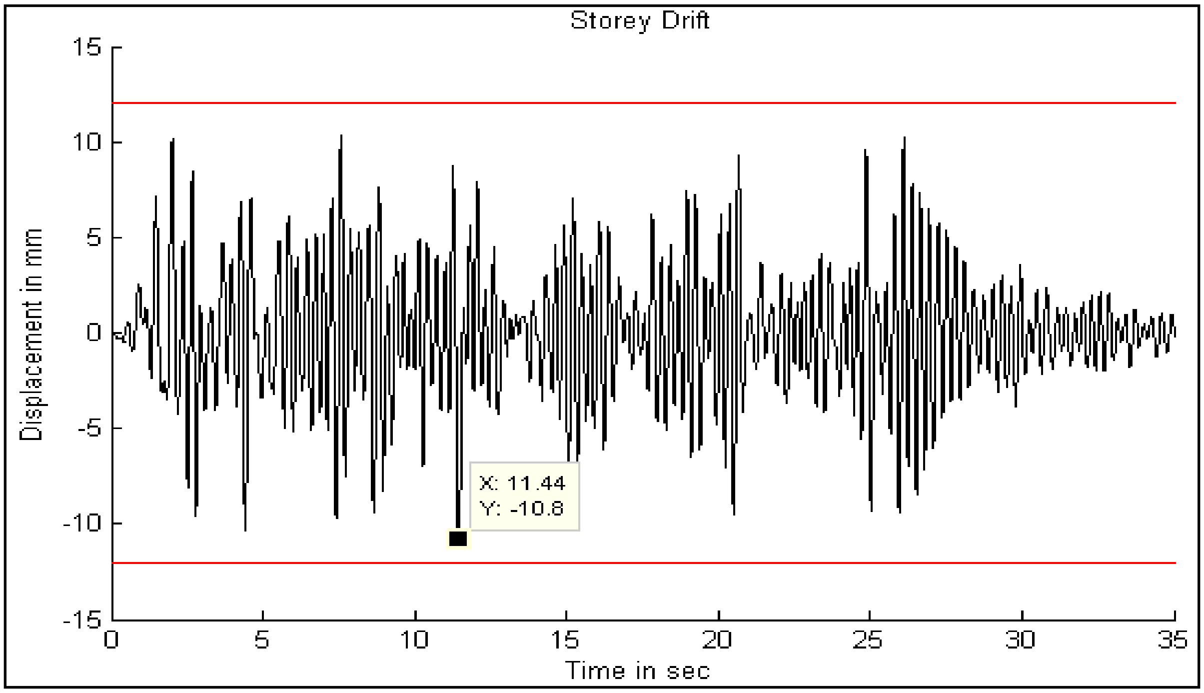


Fig. 4.52 Storey drift vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.4 m)

Table 4.21 Comparison of predicted maximum inter storey drift (mm) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Maximum storey drift (mm)	% Decrease
0.25 x 0.3	4.66	14.68	-
0.25 x 0.35	11.48	12.66	13.78
0.25 x 0.4	11.44	10.8	26.43

The time history of base shear is obtained and presented in figures 4.53-4.55. The maximum base shear is obtained from the time history plot and tabulated in Table 4.22. It is observed that the maximum base shear decreases with strengthening the ground floor columns.

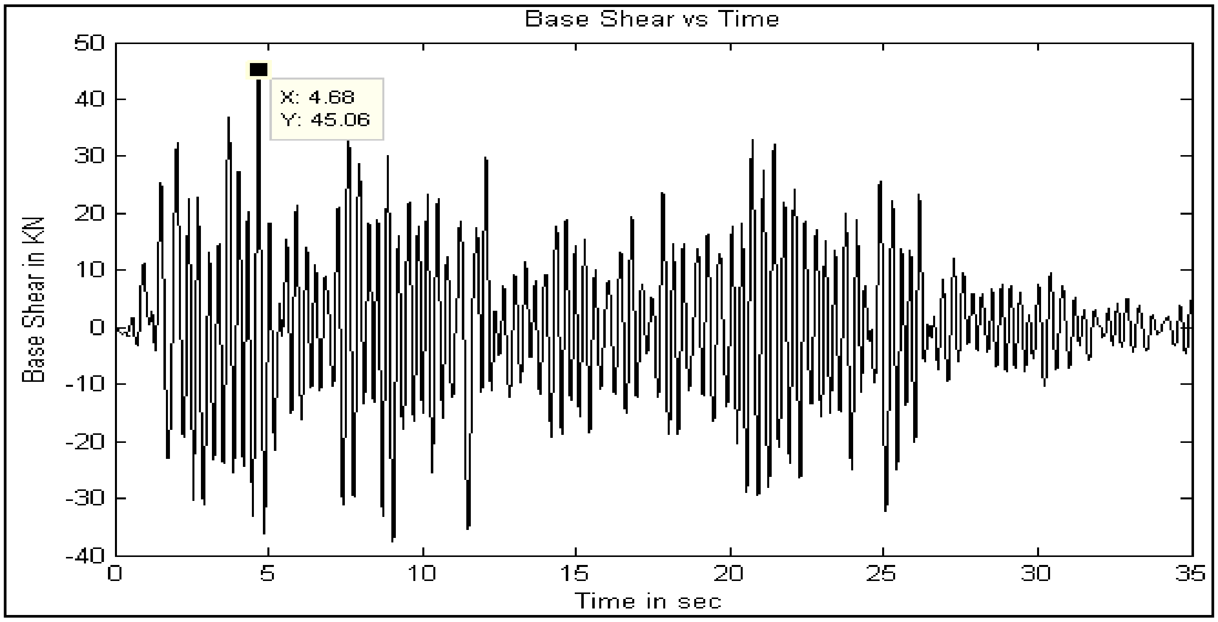


Fig. 4.53 Base shear vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

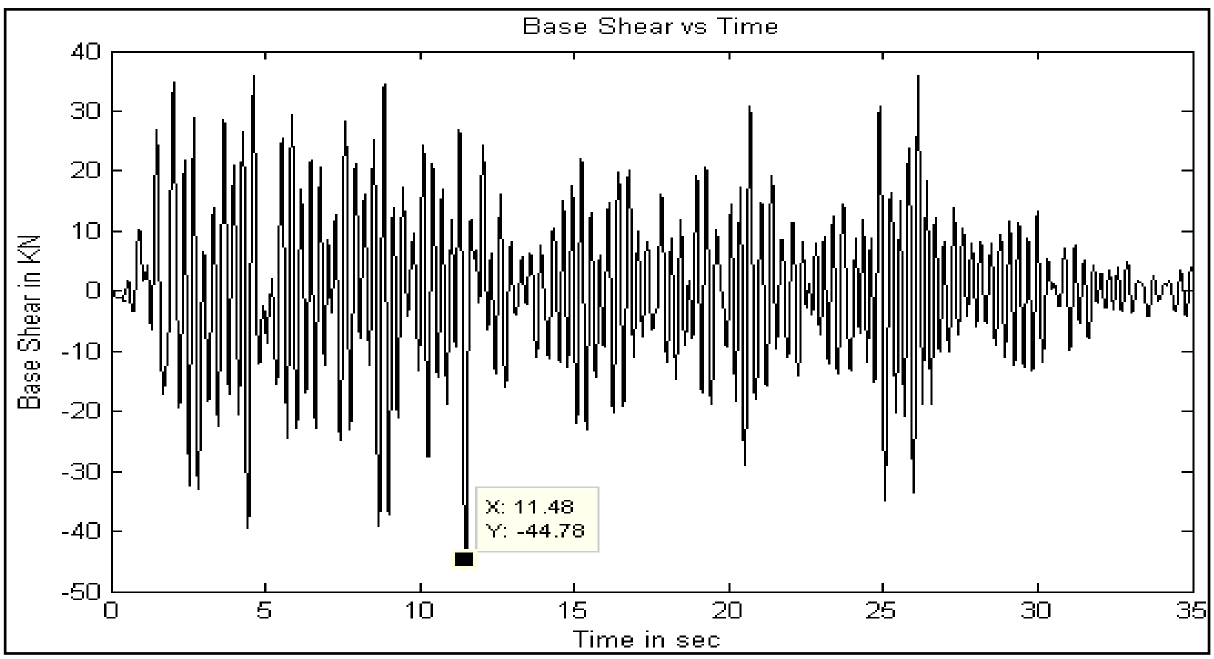


Fig. 4.54 Base shear vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

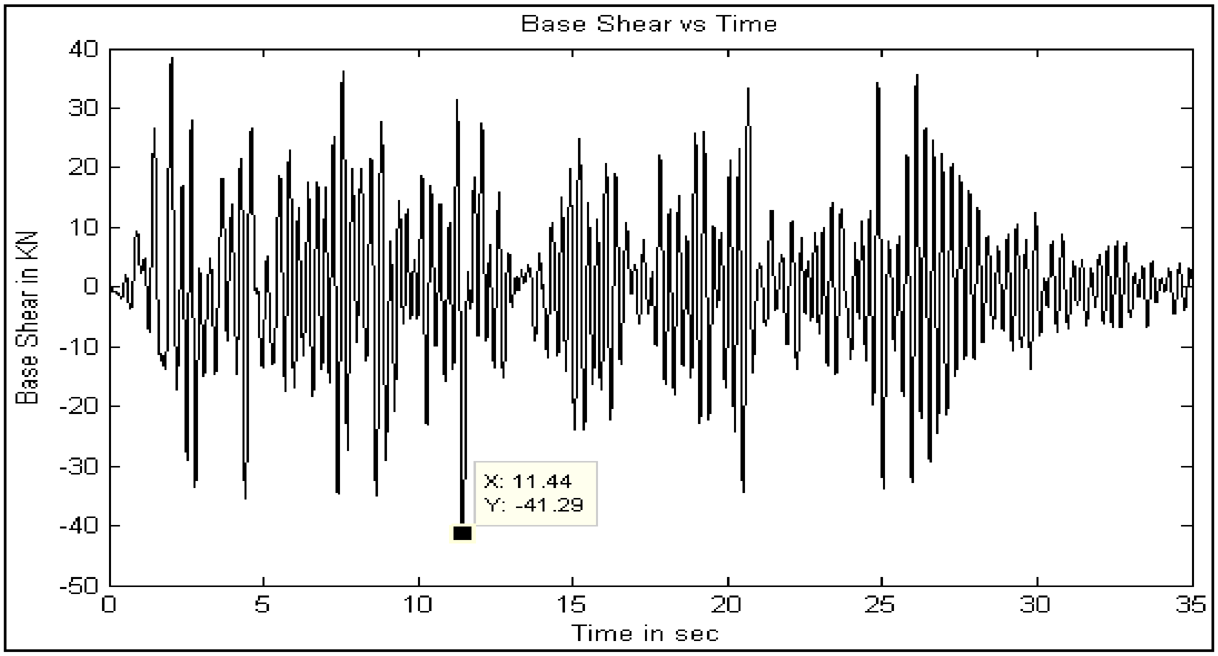


Fig. 4.55 Base shear vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.4 m)

Table 4.22 Comparison of predicted maximum base shear (kN) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Maximum base shear (kN)	% Decrease
0.25 x 0.3	4.68	45.06	-
0.25 x 0.35	11.48	44.78	0.62
0.25 x 0.4	11.44	41.29	8.36

The time history of overturning moment is obtained and presented in figures 4.56-4.58. The maximum overturning moment is obtained from the time history plot and tabulated in Table 4.23. It is observed that the maximum overturning moment decreases with strengthening the ground floor columns.

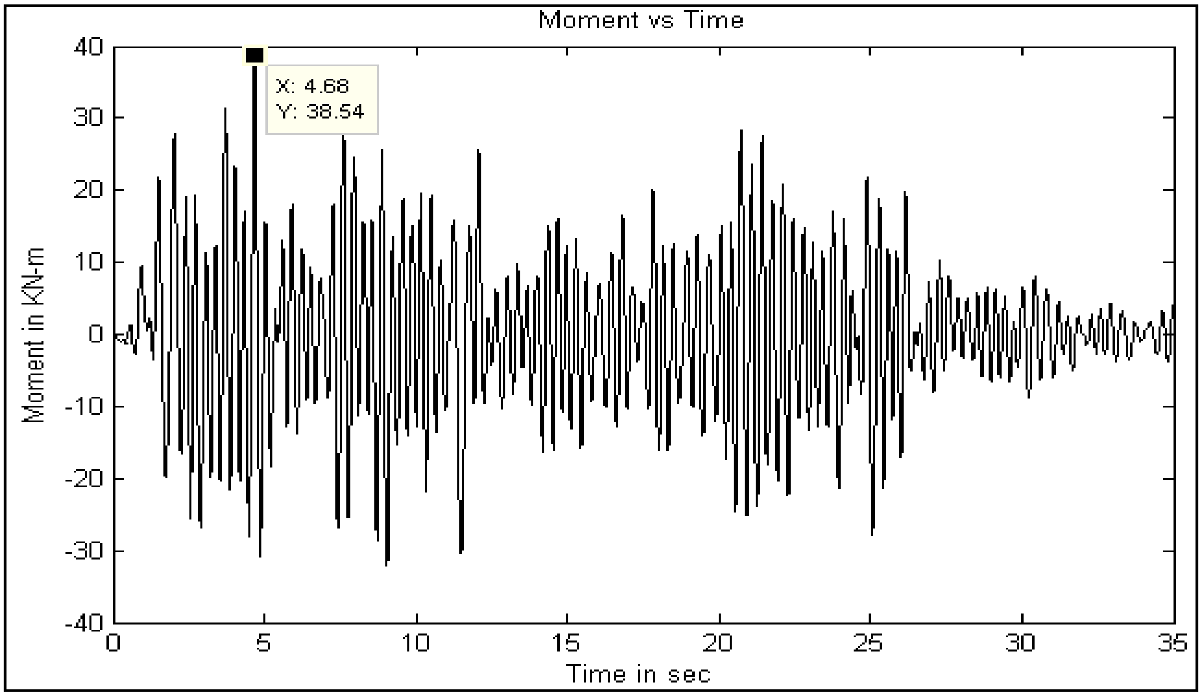


Fig. 4.56 Overturning moment vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

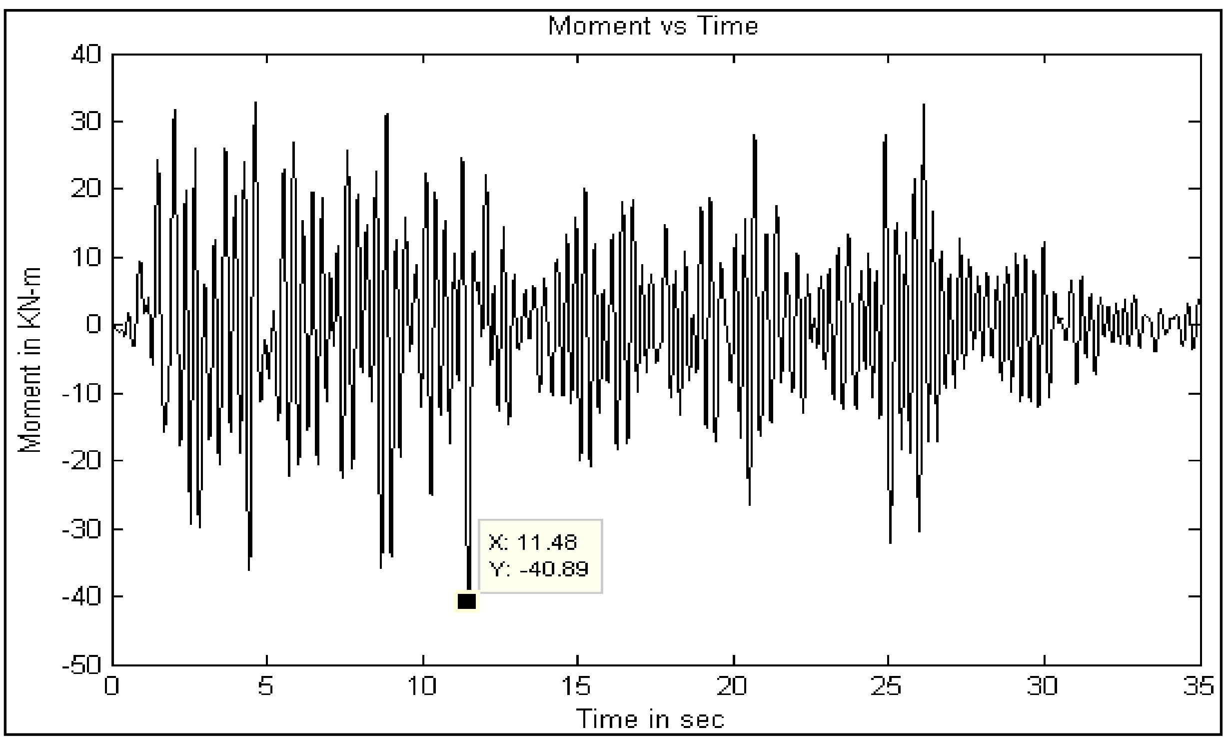


Fig. 4.57 Overturning moment vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

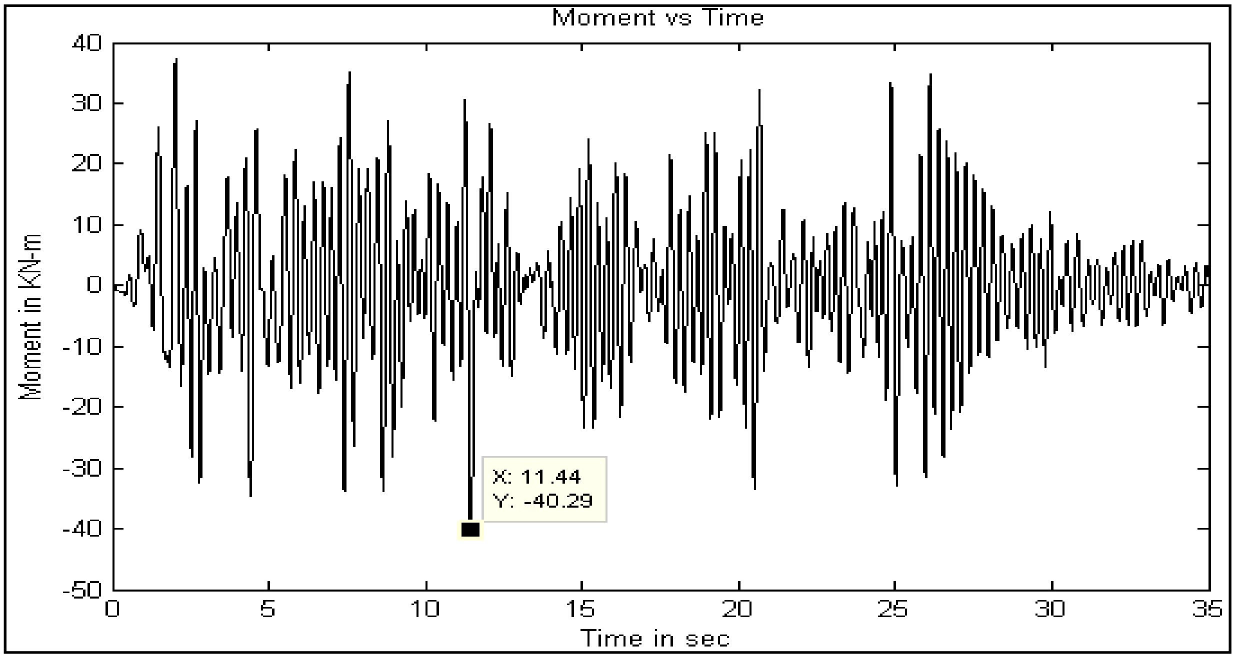


Fig. 4.58 Overturning moment vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.4 m)

Table 4.23 Comparison of predicted maximum overturning moment (kN-m) of the 2D concrete frame with floating column with size of ground floor column in increasing order

Size of ground floor column (m)	Time (sec)	Maximum overturning moment (kN-m)	% Increase
0.25 x 0.3	4.68	38.54	-
0.25 x 0.35	11.48	40.89	6.09
0.25 x 0.4	11.44	40.29	4.54

Example 4.9

In this example the same problem in Example 4.7 is analyzed under Elcentro(EW) earthquake time history data. The time history of displacement is obtained and presented in figures 4.59-4.60. The maximum displacement is obtained from the time history plot and tabulated in Table 4.24. It is observed that the maximum displacement decreases with strengthening the ground floor columns.

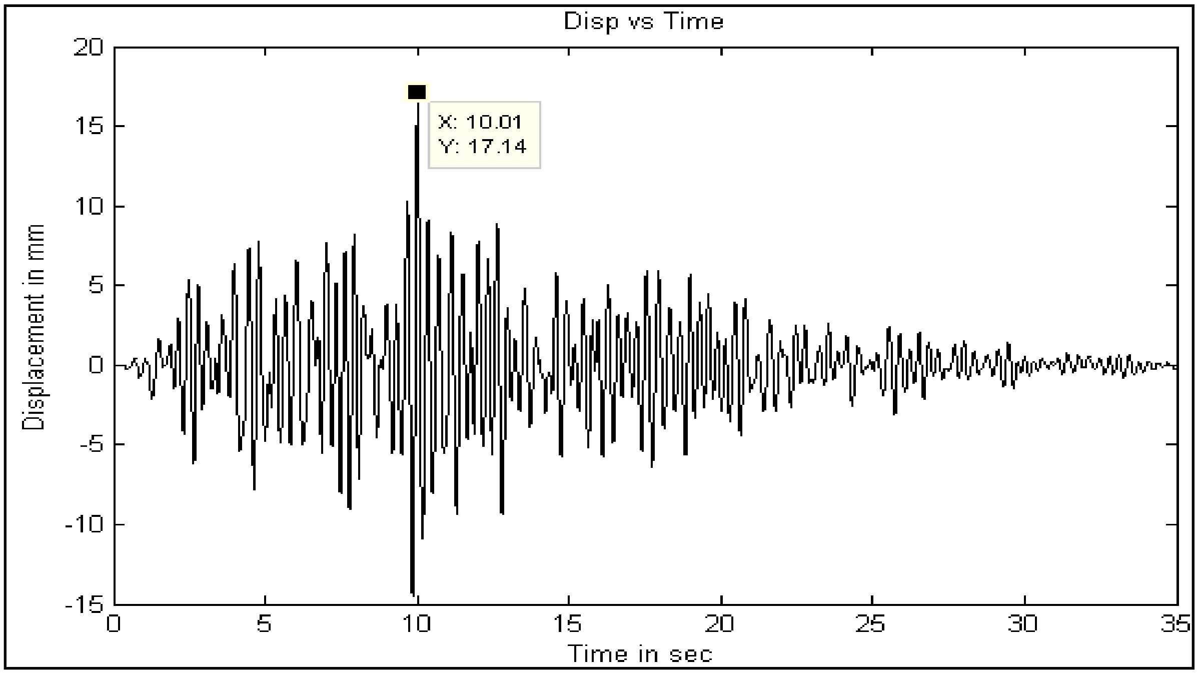


Fig. 4.59 Displacement vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

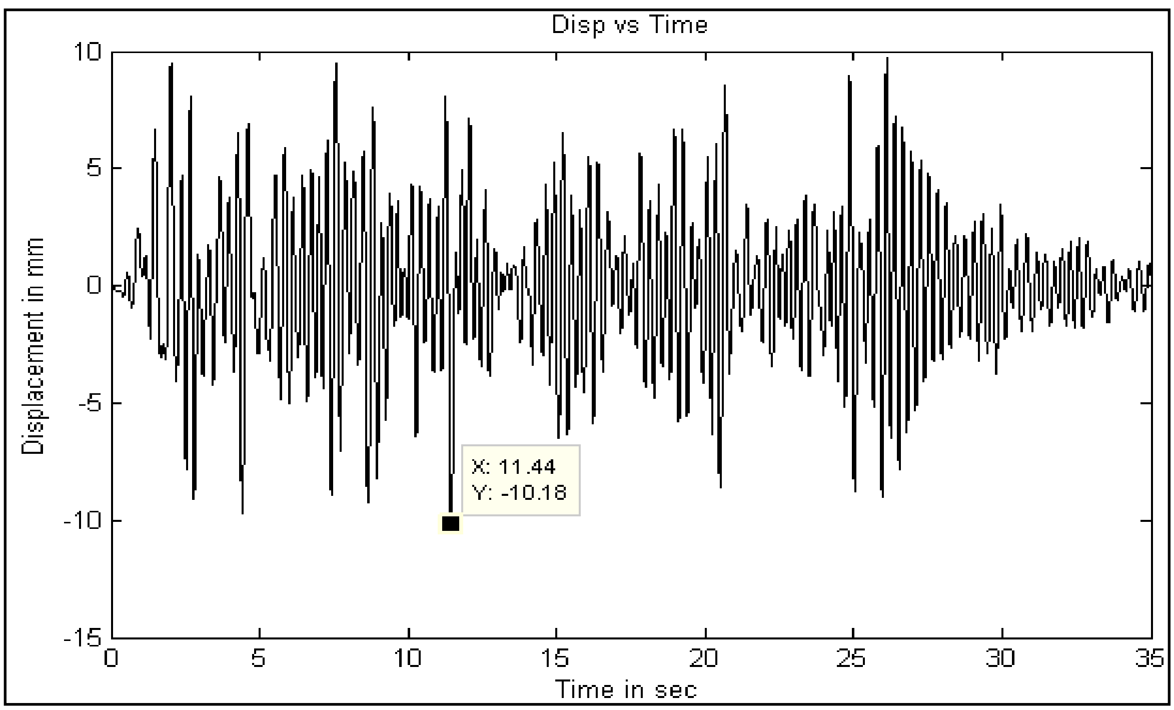


Fig. 4.60 Displacement vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

Table 4.24 Comparison of predicted maximum top floor displacement (mm) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Max displacement (mm)	% Decrease
0.25 x 0.3	4.66	13.61	-
0.25 x 0.35	11.44	10.18	25.2

The time history of inter storey drift is obtained and presented in figures 4.61-4.62. The maximum inter storey drift is obtained from the time history plot and tabulated in Table 4.25. It is observed that the maximum inter storey drift decreases with strengthening the ground floor columns.

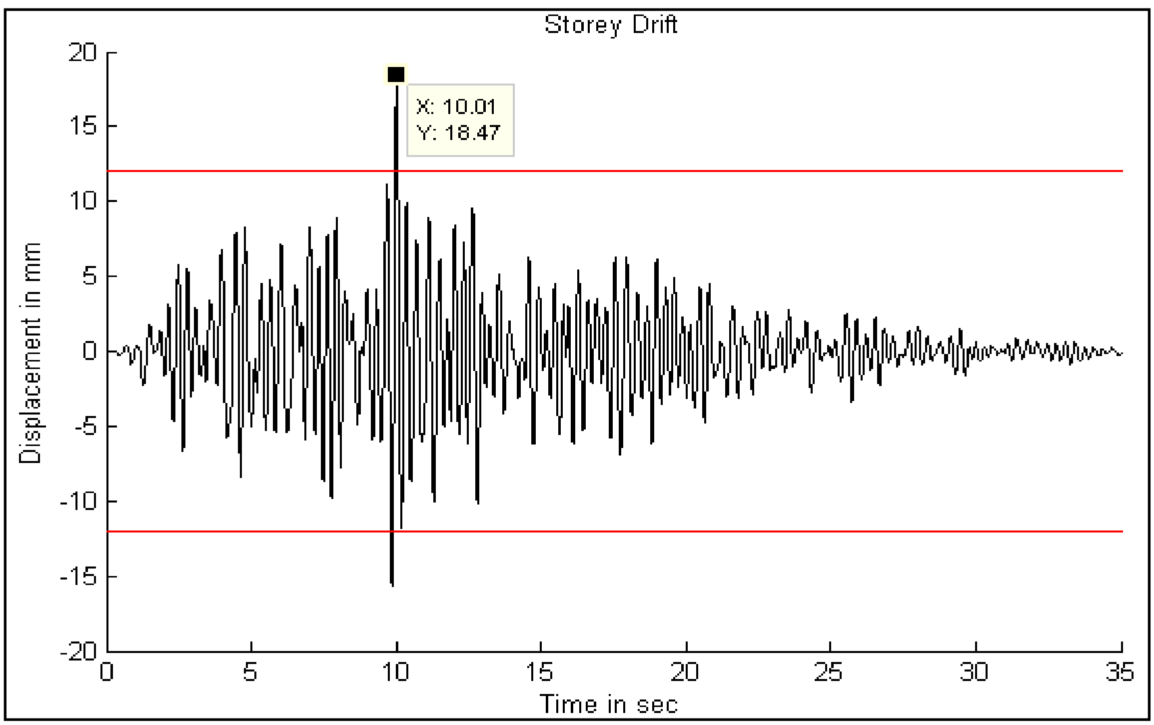


Fig. 4.61 Storey drift vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

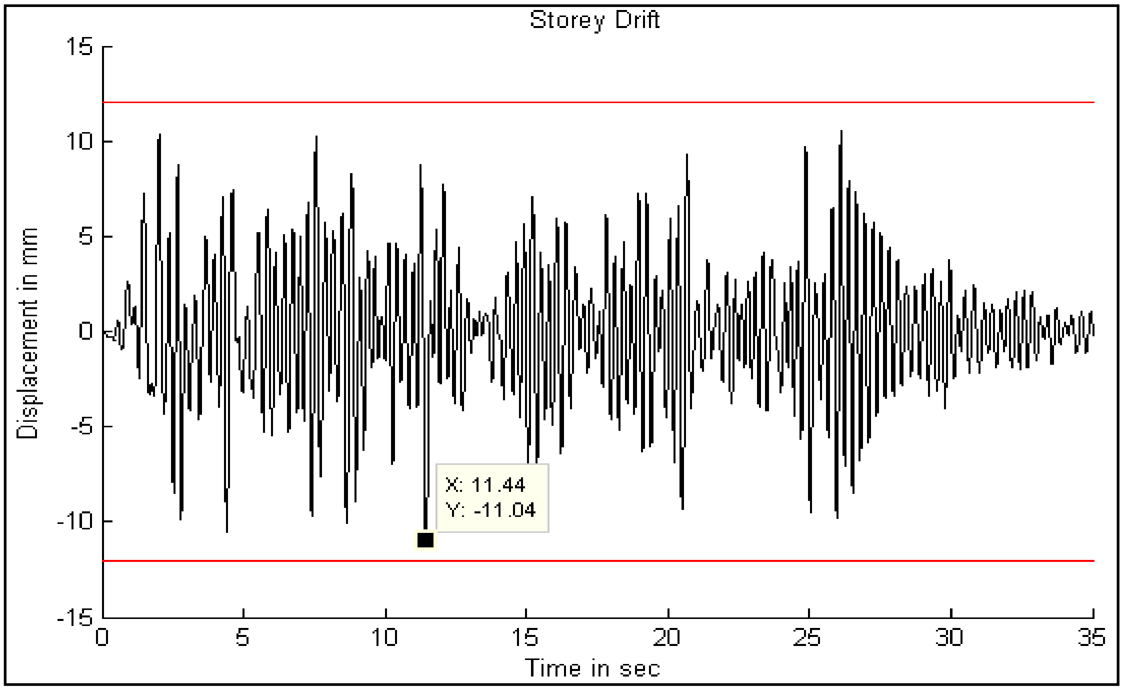


Fig. 4.62 Storey drift vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

Table 4.25 Comparison of predicted maximum inter storey drift (mm) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum storey drift (mm)	% Decrease
0.25 x 0.3	4.66	14.68	-
0.25 x 0.35	11.44	11.04	24.79

The time history of base shear is obtained and presented in figures 4.63-4.64. The maximum base shear is obtained from the time history plot and tabulated in Table 4.26. It is observed that the maximum base shear decreases with strengthening the ground floor columns.

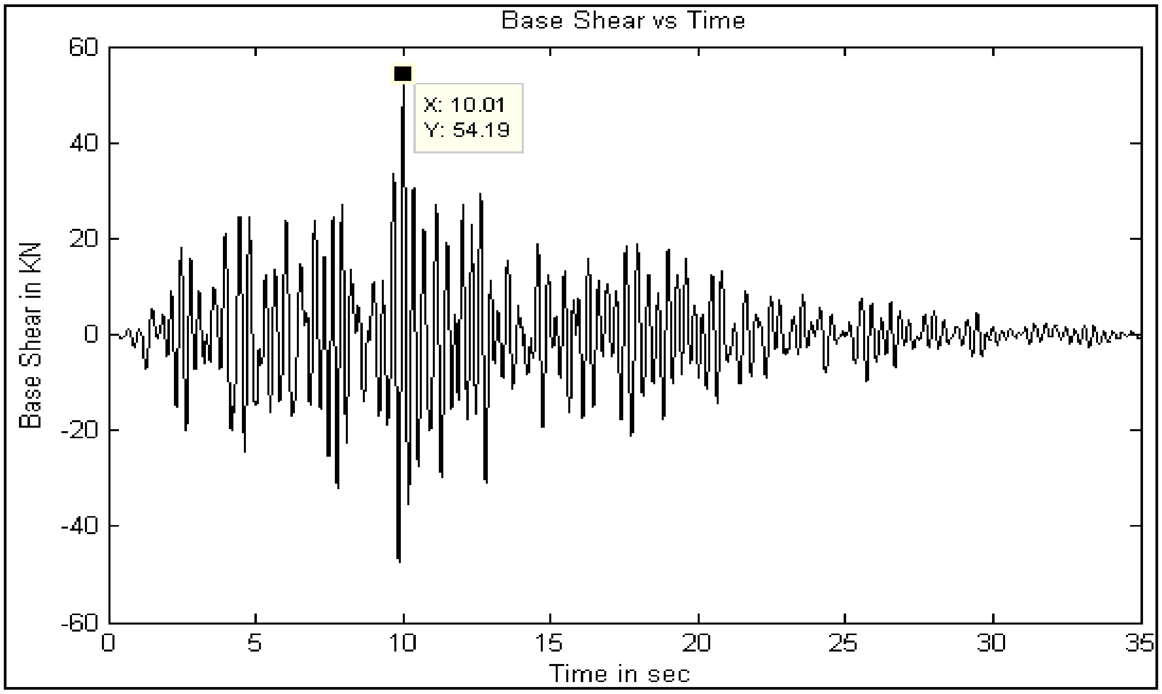


Fig. 4.63 Base shear vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

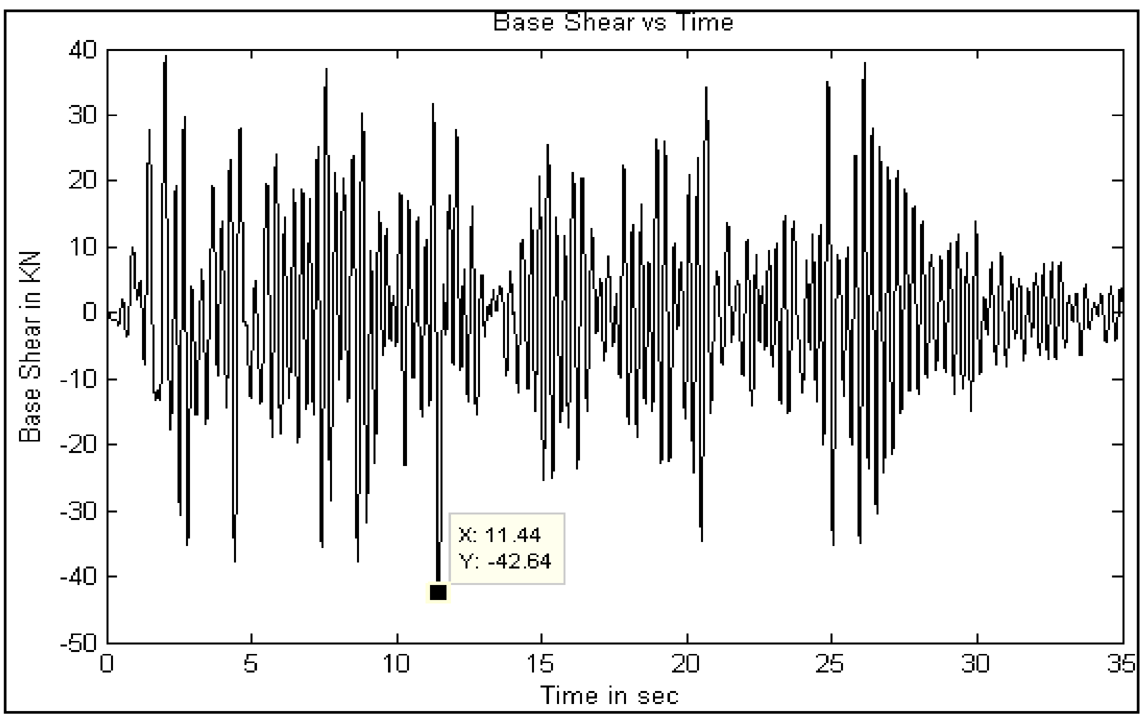


Fig. 4.64 Base shear vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

Table 4.26 Comparison of predicted maximum base shear (kN) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum base shear (kN)	% Decrease
0.25 x 0.3	4.68	45.06	-
0.25 x 0.35	11.44	42.64	5.37

The time history of overturning moment is obtained and presented in figures 4.65-4.66. The maximum overturning moment is obtained from the time history plot and tabulated in Table 4.27. It is observed that the maximum overturning moment decreases with strengthening the ground floor columns.

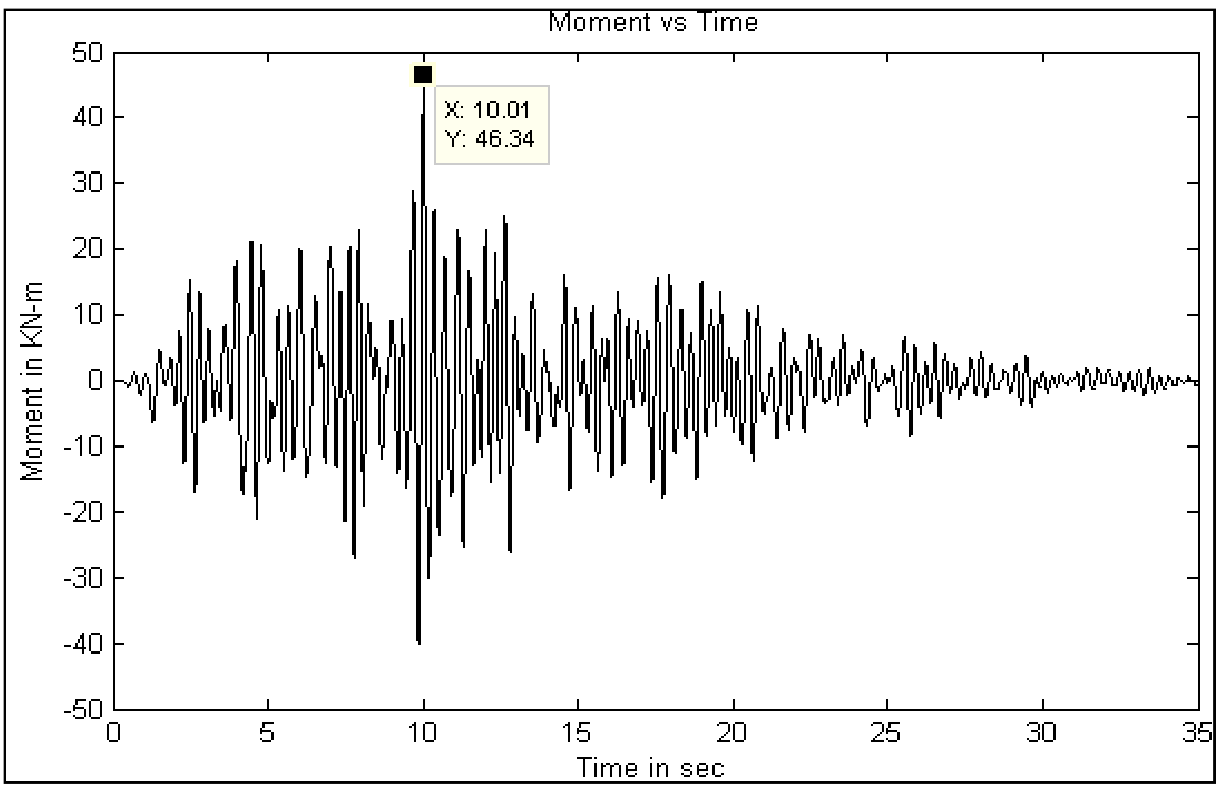


Fig. 4.65 Overturning moment vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.3 m)

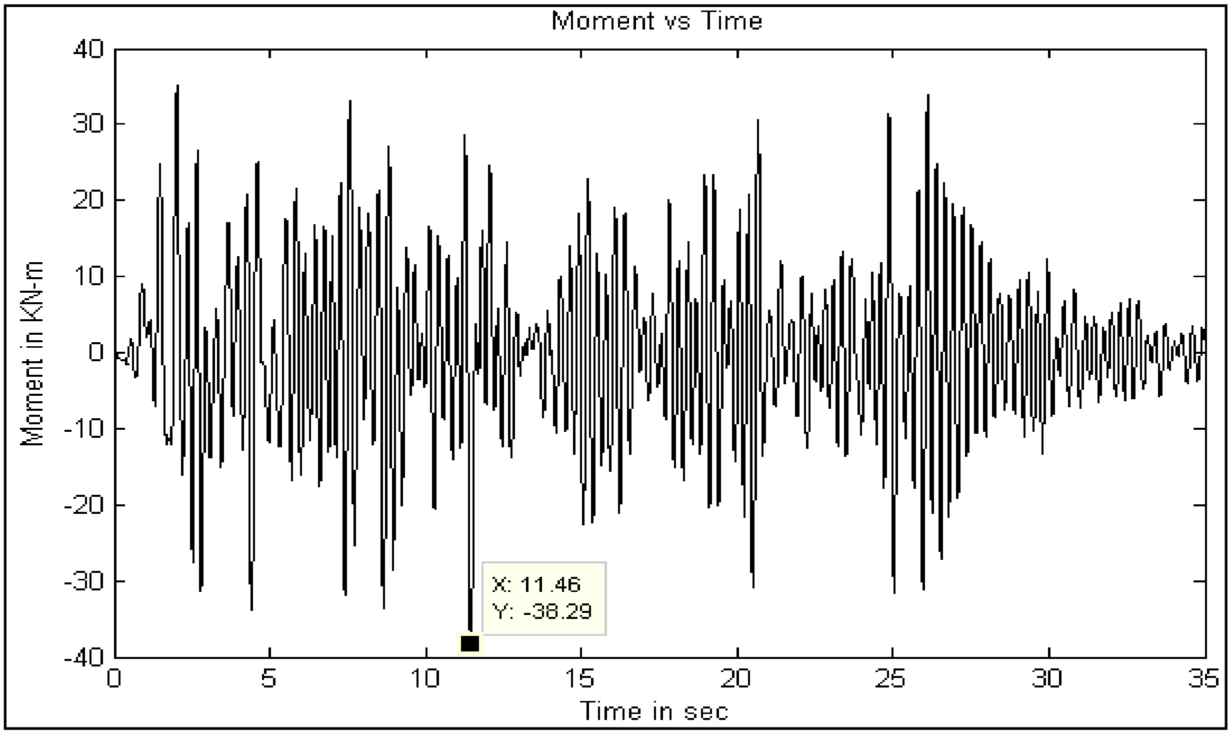


Fig. 4.66 Overturning moment vs time response of the 2D concrete frame with floating column under Elcentro time history excitation (Column size- 0.25 x 0.35 m)

Table 4.27 Comparison of predicted maximum overturning moment (kN-m) of the 2D concrete frame with floating column with size of both ground and first floor column in increasing order

Size of ground and first floor column (m)	Time (sec)	Maximum overturning moment (kN-m)	% Decrease
0.25 x 0.3	4.68	38.54	-
0.25 x 0.35	11.46	38.24	0.78

CHAPTER 5

CONCLUSION

The behavior of multistory building with and without floating column is studied under different earthquake excitation. The compatible time history and Elcentro earthquake data has been considered. The PGA of both the earthquake has been scaled to 0.2g and duration of excitation are kept same. A finite element model has been developed to study the dynamic behavior of multi story frame. The static and free vibration results obtained using present finite element code are validated. The dynamic analysis of frame is studied by varying the column dimension. It is concluded that with increase in ground floor column the maximum displacement, inter storey drift values are reducing. The base shear and overturning moment vary with the change in column dimension.

CHAPTER 6

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